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STATIC CALCULATION OF A 20,4 m STEEL BRIDGE FOR PEDESTRIAN USE

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Thanks

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A mi hermano por enseñarme el apasionante y bonito mundo de la ingeniería.

A mi padre por inculcarme a nunca dejar de ser curioso e investigar las cosas.

A mi madre por darme la oportunidad de llegar hasta aquí y seguir teniendo plena confianza en mí, pese a los todos quebraderos de cabeza que le he dado.

STATIC CALCULATION OF A 20,4 m STEEL BRIDGE FOR PEDESTRIAN USE

CÁLCULO ESTÁTICO DE UN PUENTE DE 20,4 m DE ACERO PARA USO PEATONAL

Key words: Construction, Steel constructions, Bridge, Dimensioning
Palabras clave: Construcción, Construcciones de Acero, Puente, Dimensionamiento

Summary

The diploma thesis covers the static calculation of the steel bridge for pedestrians. This bridge is 20.4 meters in length and 2.6 meters in walking width. The project task is static analysis and dimensioning of the bridge elements. The static 3D analysis is made with the AUTODESK Software Robot Structural Analysis Professional. Dimensioning is carried out in accordance with European standards of Eurocode. The location of the bridge is Valencia (Spain).

Resumen

El proyecto consiste en el cálculo estático de un puente de acero para uso peatonal. Este tiene una longitud de 20,4 m y un ancho de 2,6 metros. El proyecto se desarrolla mediante un cálculo estructural estático y un posterior dimensionamiento manual. El cálculo estático se realiza con el uso de la herramienta AUTODESK Software Robot Structural Analysis Professional, así como el modelado 3d. El dimensionamiento se realiza acorde a los Estándares Europeos del EUROCÓDIGO. La localización del puente es Valencia (España).

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SYMBOLS USED

M – Bending Moment

N – Axial force

V – Shear force

f_y – Yield strength

ε – Specific deformations

t_w – Web thickness

t_f – Flange thickness

M_{cr} – Critical bending moment to buckling

G – Steel shear module

E – Module of elasticity

I – Moment of Inertia

i – Radius inertia

λ – Buckling coeficient

γ_{M0} – Safety factor of the material

γ_{M1} – Partial safety factor of resistance

ABBREVIATIONS USED

EN-European standard

UNE-Una Norma Europea (Spanish Eurocode)

SLS- Serviceability Limit State

ULS- Ultimate Limit State

AN-Spanish National Annex

1 INTRODUCTION

The project task describes the design, analysis and dimensioning of a steel bridge for pedestrians with a length of 20,4m (6+8,4+6) and a width of 2,6 m with the unions. The Bridge consists of hot-rolled profiles, which are made according to Eurocode (EN) standards. The walkable area will be made by wooden slabs.

Three different static cases were discussed in the bridge analysis. From a static point of view, the second one was chosen, as it was the easiest. The weight analysis was carried out with the Eurocode 1 standard (EN 1). The total own weight of the overall structure and the other elements contained in the bridge was also considered. The static analysis was calculated with the AUTODESK software Robot Structural Analysis Professional program.

Once the static analysis is done, each element is separately dimensioned depending on the weight being handled. Dimensioning was carried out according to the procedure prescribed by Eurocode standards. In the dimensioning of steel elements, we used Eurocode 3 (EN 3) and Eurocode 5 (EN 5) for wooden walking elements. The dimension has been calculated referenced to the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2 BRIDGE DESIGN

The bridge considered is only for a pedestrian use. It has a range of 20,4 m and a width of 2,6 m. The entire surface is covered by wooden slats of 2,6 m length, a width of 0,25 m and a thickness of 6 cm, the wood is a conifer type C30. Other two parts of 6 m are supported by two vertical columns of 1,75m and a couple of diagonal beams. The main part of the bridge, which is 8 m long, is in the middle.

Two main beams with a total length of 20,4 m are designed from HEA 200 profile with a distance of 2,6 m. Over them the wooden slabs are going to be placed. Every part is crossed by a couple HEB 120 profile in order to reinforce the structure. At the beginning, end and above the main beams HEA 100 profiles are placed, these last ones give stability and strength to the structure. All steel elements are steel-quality S275 and are protected with a stainless threatment.

The bridge have a 1,2 m high balustrade consisting of steel tubes with a diameter of 30 mm and a wall thickness of 4 mm, this fence is supported by HEB 120 vertical columns that are 50 cm projected from each other main beam, these vertical columns are 1,7 m separate between them. Additionally, the fence is divided in 0,2 m spaces with a 6 mm steel wire. It is connected to the structure with steel bolts in order to prevent a failure for bending stresses.

2.1 DRAWINGS OF THE BRIDGE

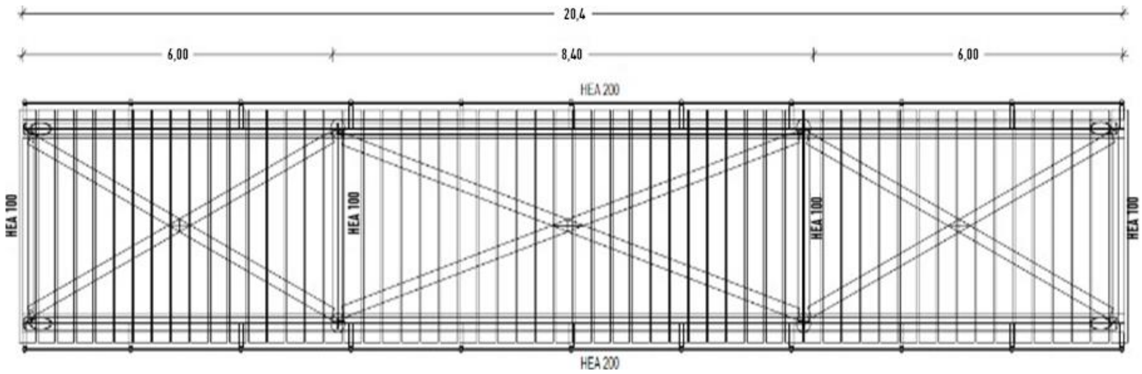


Image 1: Top view

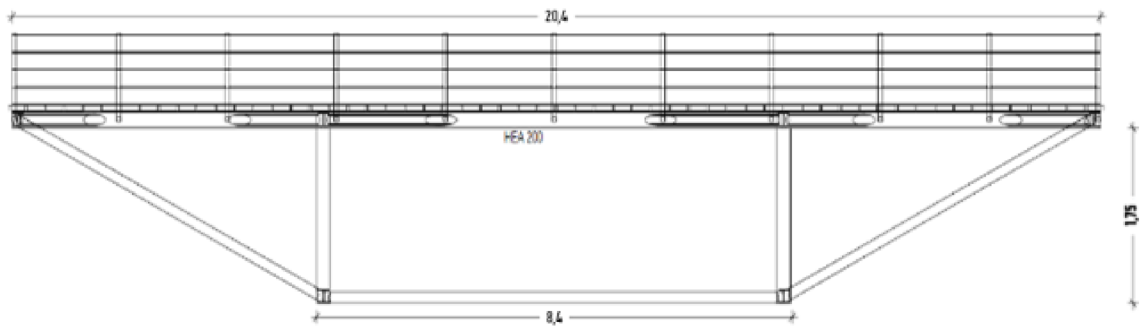


Image 2: Top view

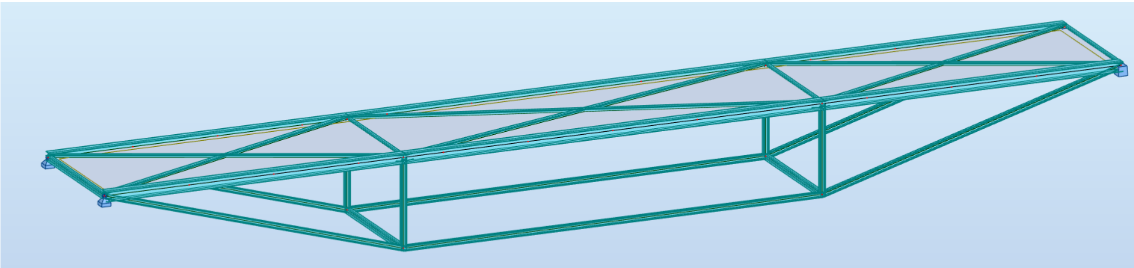


Image 3: Perspective view

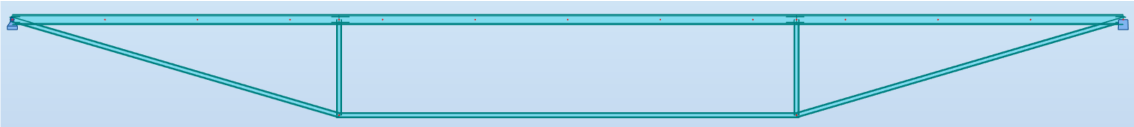


Image 4:Lateral view

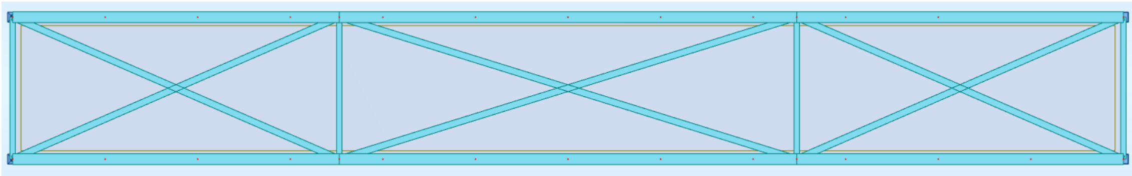


Image 5: Top view

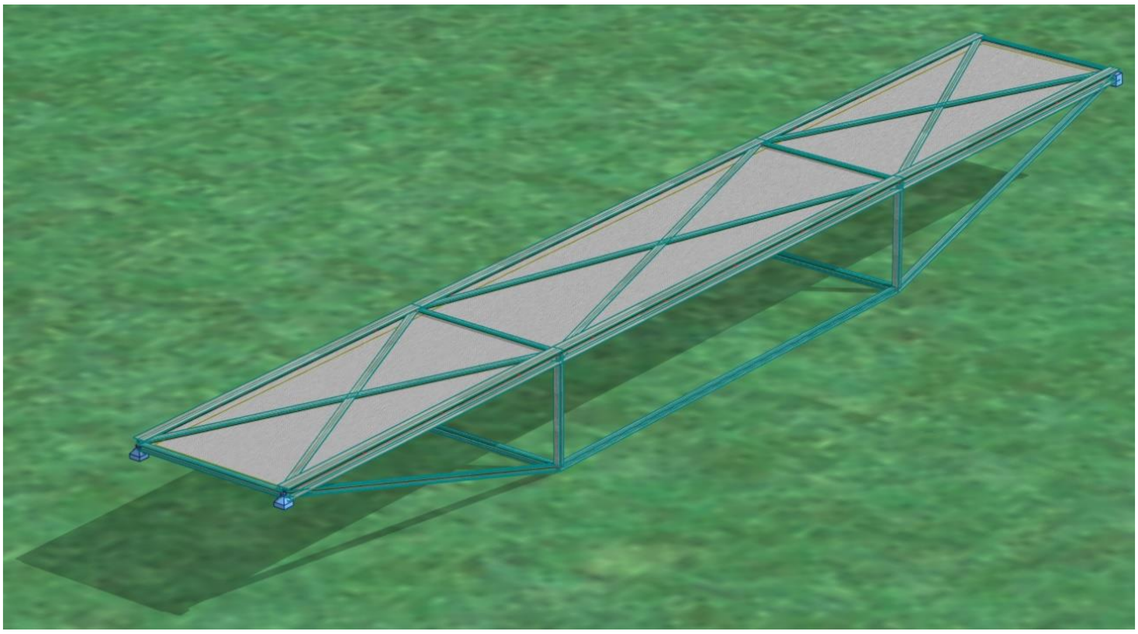


Image 7: Rendered view

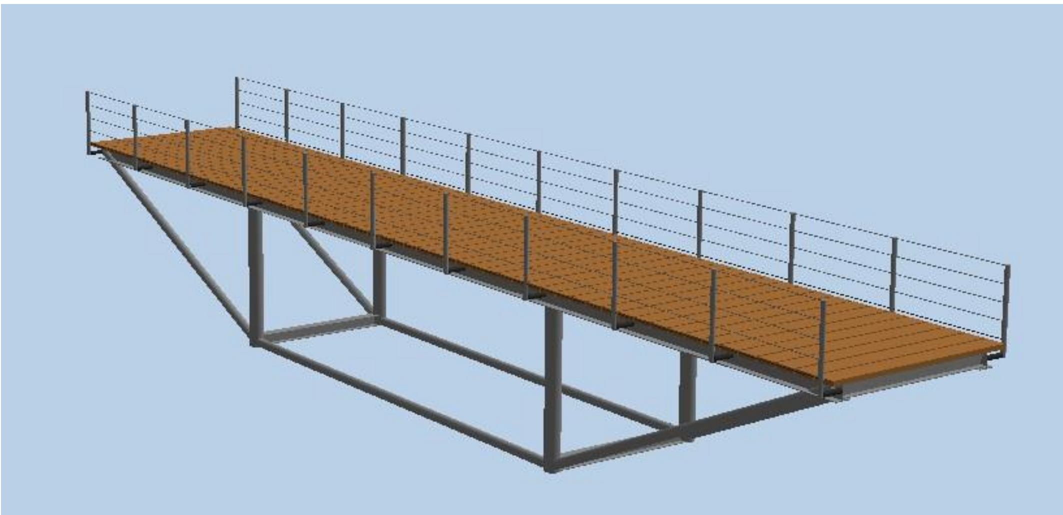


Image 6: Rendered view with all the elements

3 LOADS

3.1 CONSTANT WEIGHT

Despite constant weight is not as relevant for dimensioning than the wind, in order to get the most accurate values, it is going to be shown in two tables. The reason is that almost every structural calculation program offers the possibility of disabling the self weight of the structural elements.

The first table measures only the basic structural part without the wooden slabs and the fence, this self-weight of the structure is automatically calculated inside the Autodesk Robot program.

Table 3.1.1: Constant Weight of Structure

Profile	Quality	Quantity	Weight (kg/m)	Total length (m)	Weight * length (kg)	Load (kN)
HEA 200	S 275	2	42,3	40,8	1725,84	17,25
HEA 100	S 275	16	16,7	64,4	1075,48	10,75
HEB 120	S 275	6	26.7	43,74	1167,85	11,67

Total weight of structure: 39,67 kN

The second table shows the weight of the wooden slabs and the fence. According to table A.3 in Eurocode 1-1 the value of load for C30 is $\gamma = 4,6\ kN/m^3$, the thickness of the wooden slabs is $e = 0,06\ m$

Table 3.1.2: Constant Weight of floor and fences

Profile	Quality	Quantity	Weight (kg/m)	Total length (m)	Weight * length (kg)	Load (kN)
2,6 x 0,25 x 0,06 Wooden slab	C30	82	-	20,4	-	14,64
6 mm steel wire	S 275	10	0,15	204	30,6	0,30
HEB 120	S 275	26	26,7	44,2	1180,14	11,80

Total weight of the floor and fences: 26,74 kN

Them the weight that main beams have to carry is:

Case a)

In this case the weight of the basic structural elements, feces and floor are placed as distributed loads over the two main beams.

$$\frac{39{,}67\ kN + 26{,}74kN}{20{,}4\ m \cdot 2} = 1{,}62\ kN/m$$

(3.1.1)

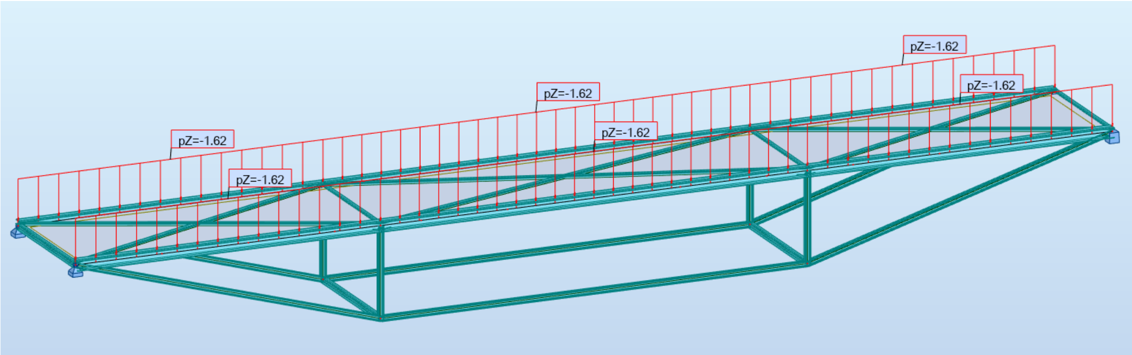


Image 8: Weight of basic structural elements as distributed load

Case b)

In this case the shelf-weight of the basic structural steel elements has been calculated by the software, while the weight of the feces and floor has been placed as distributed loads over the two main beams.

$$\frac{26{,}74kN}{20{,}4\ m \cdot 2} = 0{,}66\ kN/m$$

(3.1.2)

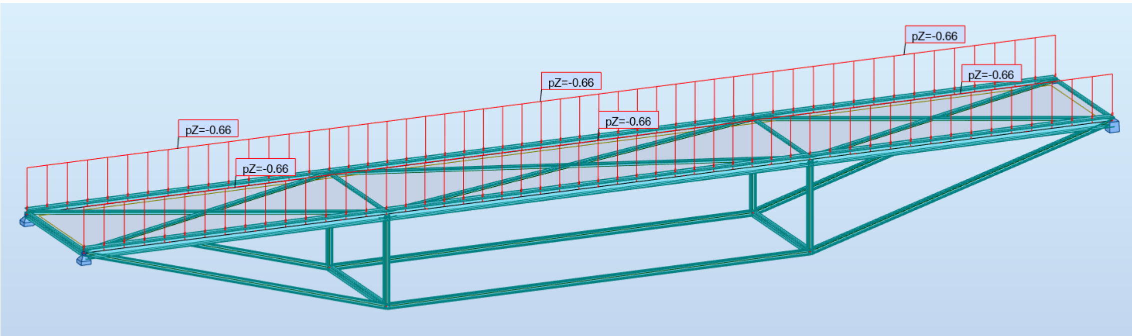


Image 9: Weight of basic structural elements calculated by software

In practice both calculations end in a quite similar results, but the second case is going to be used in this work, because it reflects the reality in a more accurate way.

3.2 VARIABLE WEIGHT

3.2.1 SNOW LOAD

The snow load was calculated according to the standard UNE-EN 1991-1-3:2018 according to equation 5.1, which applies to permanent or temporary project status.

Our bridge is located in Valencia at an altitude of A = 15m. It belongs to Zone 5.

$$s = \mu_i * C_e * C_t * S_k \tag{3.2.1.1}$$

- μ_I – Snow weight design coefficient
- C_E – Exposure coefficient
- C_T –Heat coefficient
- S_k – The characteristic weight of snow on the ground

C_e = 1,0; C_t = 1,0; μ_i = 0,8

S_k is directly taken from the table AN.2 in the Spanish National Annex 4.1 UNE-EN 1991-1-3:2018 according to the zone 5 in figure AN.1.

$$Sk = 0{,}2\ \frac{KN}{m^2} \tag{3.2.1.2}$$

Image AN.1 – Climatic zones

Image 10: Map of snow zones AN.1 Spanish National Annex 4.1 (UNE-EN 1991-1-3:2018)

Table AN.2 – Snow load over a horizontal surface, s_k $[kN/m^2]$

Altitud [m]	Zona climática de invierno (según figura AN.1)						
	1	**2**	**3**	**4**	**5**	**6**	**7**
0	0,3	0,4	0,2	0,2	0,2	0,2	0
200	0,5	0,5	0,2	0,2	0,3	0,2	0
400	0,6	0,6	0,2	0,3	0,4	0,2	0
500	0,7	0,7	0,3	0,4	0,4	0,3	0
600	0,9	0,9	0,3	0,5	0,5	0,4	0
700	1,0	1,0	0,4	0,6	0,6	0,5	0
800	1,2	1,1	0,5	0,8	0,7	0,7	0
900	1,4	1,3	0,6	1,0	0,8	0,9	0
1000	1,7	1,5	0,7	1,2	0,9	1,2	0
1200	2,3	2,0	1,1	1,9	1,3	2,0	0
1400	3,2	2,6	1,7	3,0	1,8	3,3	0
1600	4,3	3,5	2,6	4,6	2,5	4,3	0
1800	4,3	4,6	4,0	4,6	2,5	4,3	0

Image 11: Table of S_k values according to height Spanish National Annex 4.1 (UNE-EN 1991-1-3:2018)

With all the necessary values we can calculate the weight of snow.

$$s = \mu_i \cdot C_e \cdot C_t \cdot S_k \quad (3.2.1.3)$$

$$s = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 0{,}2 = 0{,}16\ \frac{KN}{m^2} \quad (3.2.1.4)$$

In order to calculate the continuous weight of the snow on the beam, the value of snow weight is multiplied by the width of the surface area and divided by the number of main beams.

$$q_s = \frac{s \cdot width}{2} = \frac{0{,}16\frac{KN}{m} \cdot 2{,}6\ m}{2} = 0{,}208\ KN/m \quad (3.2.1.5)$$

3.2.2 WIND LOAD

The wind must be calculated according to the EUROCODE standards, they are specified in the UNE-EN 1991-1-4:2018// EN 1991-1-4:2005; for that reason the following math formulas will be referenced to them.

The wind on the bridge operates in three directions:

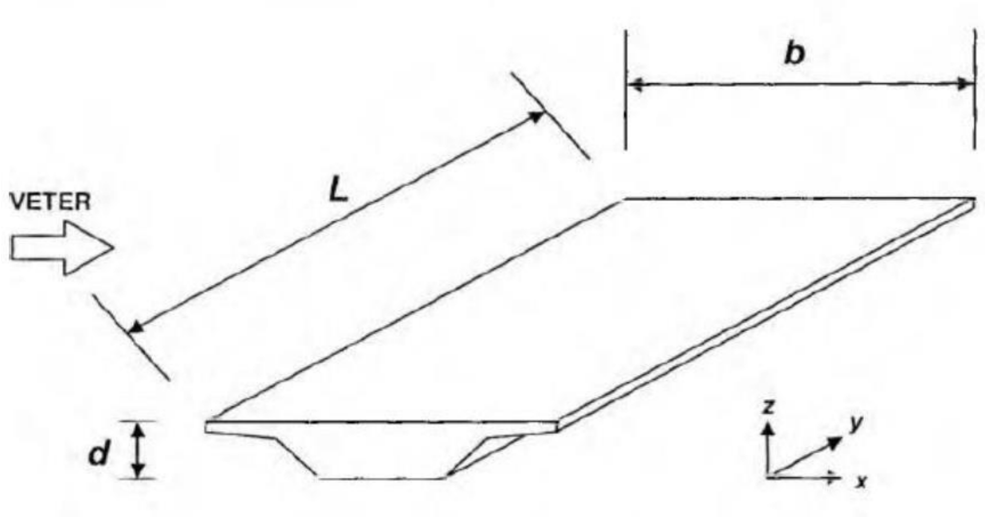


Image 12: Directions of wind actions on bridges EN 1991-1-4:2005// EN 1991-1-4:2005

- In x direction perpendicular to the construction
- In y direction longitudinal to the structure
- In z direction vertical to the walkable area

First of all, the basic wind velocity (v_b) has to be calculated following expression 4.1:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}$$
(3.2.2.1)

The national annex brings us the fundamental value of the basic wind velocity for our place (Valencia):

$$v_{b,0} = 26\, m/s$$
(3.2.2.2)

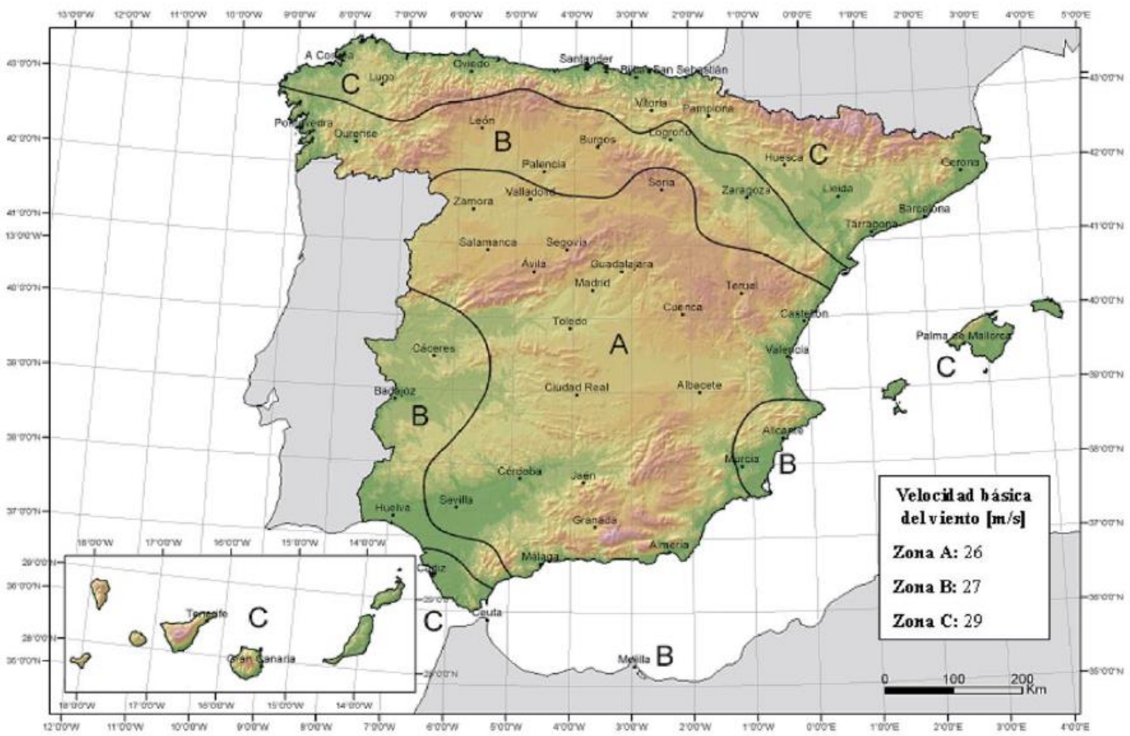


Figure AN.1 – Map of fundamental basic wind velocity $v_{b,0}$

Image 13: Table of wind zones AN.1 Spanish National Annex 4.2 (UNE-EN 1991-1-4:2018)

The recommended value for C_{dir} (directional factor) is $\mathbf{C_{dir} = 1,0}$.
Also the recommended value for C_{season} (season factor) is $\mathbf{C_{season} = 1,0}$

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} \tag{3.2.2.3}$$

$$v_b = 1 \cdot 1 \cdot 26 = 26\, m/s \tag{3.2.2.4}$$

Because our bridge construction is a normal one and it is not too demanding, since the bridge is smaller than the 40 m length, it does not require a dynamic response calculation. Thus, we can use the simplified method prescribed by one for each direction separately.

1. Direction x

$$F_w = \frac{1}{2} \cdot \rho \cdot v_b^2 \cdot C \cdot A_{ref,x} \tag{3.2.2.4}$$

Where:

- v_b - Basic Wind velocity,
- C - Wind load factor,
- $A_{ref,x}$ - Reference area,
- ρ - Air density.

Calculation of wind load factor C:

$$C = C_{fx,0} \cdot C_e(z) \tag{3.2.2.4}$$

- $C_{fx,0}$ – The bridge force coefficient in direction x
- $C_e(z)$ – Exposure factor

In order to determine the value of the force coefficient for the bridge, we need to calculate the ratio between the width of bridge **(b)** and the height of the cross-sectional structure **(b_{tot})** including the fence from the graph.
The following assumption is going to be taken, the primary beam is expected to be smaller than a HEA 300.

$$d_{tot} = h_n + h_d + h_o \tag{3.2.2.5}$$

- h_n -Total height of the primary beam HEA 300,
- h_d – Height of the boards,
- h_o – The height of the fence.

$$h_n = 0{,}29\ m;\ h_d = 0{,}06\ m;\ h_o = 1{,}2\ m \tag{3.2.2.6}$$

$$d_{tot} = 0{,}29\ m + 0{,}06\ m + 1{,}2\ m = 1{,}55\ m \tag{3.2.2.7}$$

$$\frac{b}{d_{tot}} = \frac{2{,}6}{1{,}55} = 1{,}67 \tag{3.2.2.8}$$

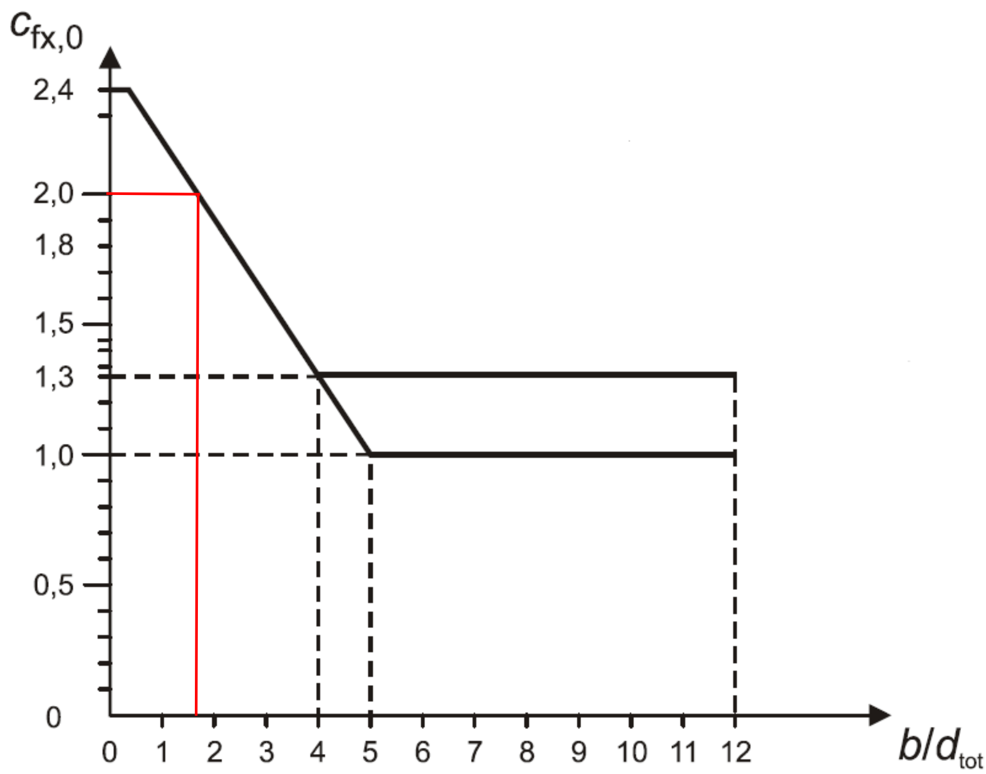


Image 14: Coefficient of force for bridges, Cfx,0 (EN 1991-1-4:2005 (E)//UNE-EN 1991-1-4:2018)

$$C_{fx,0} = 2{,}0$$

(3.2.2.9)

The exposure factor coefficient is assumed according to the second category of terrain and at a height of 6 meters.

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

Image 15: Table of terrain categories and terrain parameters

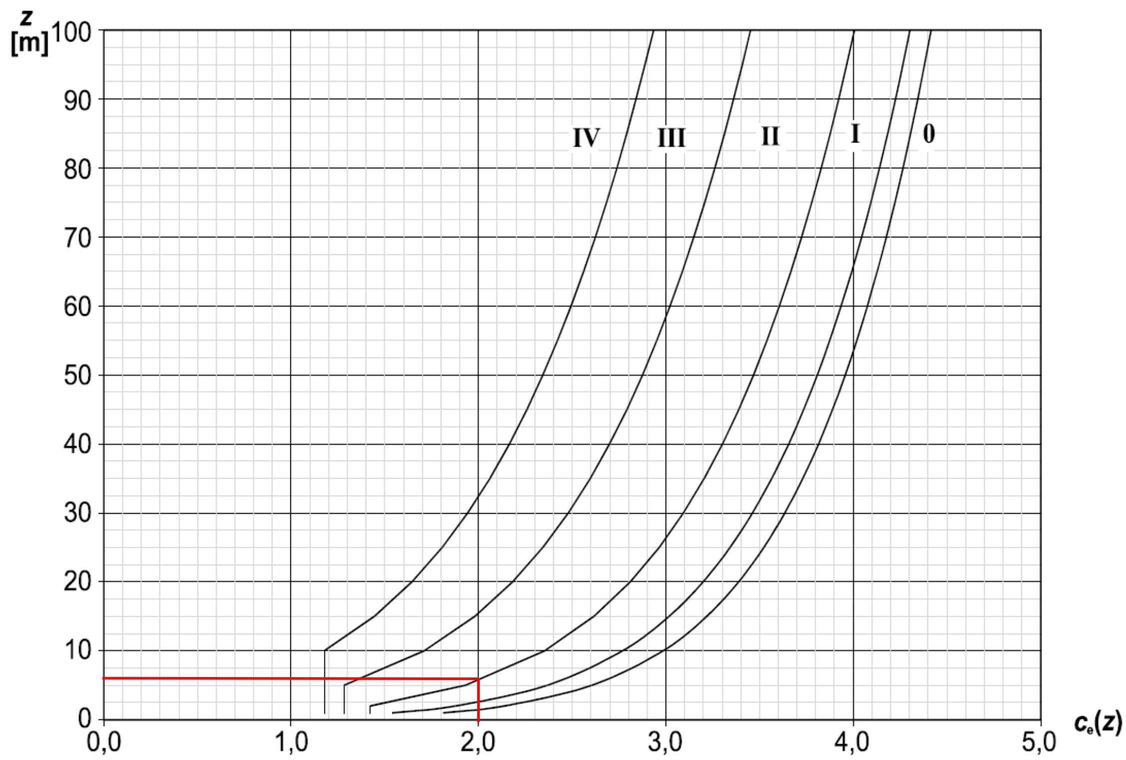


Image 16: Exposure factor Ce(z) for Co=1,0, KI=1,0

$$C_e(z) = 2{,}0 \tag{3.2.2.10}$$

Finally, we get the wind load factor C as:

$$C = C_{fx,0} \cdot C_e(z) \tag{3.2.2.11}$$

$$C = 2{,}0 \cdot 2{,}0 = 4{,}0 \tag{3.2.2.12}$$

Calculation of the Reference area, $A_{ref,x}$:

$$A_{ref,x} = d \cdot l \tag{3.2.2.13}$$

- d – The height of the Bridge Section
- l – The length of the bridge

The height of the bridge is the sum of the height of the main beam, the height of the walking boards and twice the height of the fence, as it works on both sides.

$$A_{ref,x} = (0{,}29 \text{ m} + 0{,}06 \text{ m} + 2 \cdot 1{,}2 \text{ m}) \cdot 20{,}4 \text{ m} = 57{,}528 \text{ m}^2$$

Due standard air density is 1.25kg/m^3 the force in direction x is:

$$F_{w,x} = \frac{1}{2} \cdot \rho \cdot v_b^2 \cdot C \cdot A_{ref,x}$$

(3.2.2.14)

$$F_{w,x} = \frac{1}{2} \cdot 1{,}25 \frac{Kg}{m^3} \cdot \left(26 \frac{m}{s}\right)^2 \cdot 4{,}0 \cdot 57{,}528\, m^2 = 97222\, N$$

(3.2.2.15)

$$q_{w,x} = \frac{97{,}22\, KN}{20{,}4\, m \cdot 2} = 2{,}383\, KN/m$$

(3.2.2.16)

Despite we are not taking the area of the pedestrians, this calculation is in the safety side due to twice the height of the fence was used for calculations; 2,4 m which is significantly greater that the normal size of a person. Also, vertical and diagonal beams where not consider due their small cross-section dimensions and the fact that. Also the fence was considered with full area while this is not the case.

2. Direction z:

The only difference from X direction is the wind weight factor and the reference height.

$$C = C_{f,z} \cdot C_e(z)$$

(3.2.2.17)

- $C_{f,z}$ – Coefficient of wind force in direction Z,
- $C_e(z)$ – Exposure factor.

Coefficient $C_{f,z}$ has been taken from standard EN 1, which covers the influence of the transverse slope of the lintel, the slope of the terrain and the change in the angle of direction due to turbulence. The value is positive and negative ± 0.9.

The reference area is the width of the bridge multiplied by the length.

$$A_{ref,z} = d \cdot l$$

(3.2.2.18)

$$A_{ref,x} = 2{,}6\, m \cdot 20{,}4\, m = 53{,}04\, m^2$$

(3.2.2.19)

Now, we can proceed to calculate the force in Z direction:

$$F_{w,z} = \frac{1}{2} \cdot \rho \cdot v_b^2 \cdot C_{f,z} \cdot C_e(z) \cdot A_{ref,x}$$

(3.2.2.20)

$$F_{w,z} = \frac{1}{2} \cdot 1{,}25 \frac{Kg}{m^3} \cdot \left(26 \frac{m}{s}\right)^2 \cdot 2{,}0 \cdot (\pm\, 0.9) \cdot 53{,}04\, m^2 = 40336\, N$$

(3.2.2.21)

It is necessary to consider the excentric force. According to the standard e = b/4 can be taken. The value of the force $F_{w,z}$ is divided between two main beams:

- First beam: $\frac{F_{w,z}}{4} = 10084\, N$

(3.2.2.22)

- Second beam: $\frac{3 \cdot F_{w,z}}{4} = 30252\, N$

(3.2.2.23)

$$q_{w,z} = \frac{10{,}08\, KN}{20{,}4\, m \cdot 2} = 0{,}247\, KN/m$$

(3.2.2.24)

3. Direction y:

According to the standards, the recommended values (25% $F_{w,x}$) have to be multiplied by a reduction coefficient that is calculated by the following formula:

$$1 - \left(\frac{7}{C_o \cdot \ln\left(\frac{z}{z_0}\right) + 7}\right) \cdot \phi \cdot \left[\frac{L}{L(z)}\right]$$

(3.2.2.25)

Where:

- C_o – Is the topographic factor,
- z – Is the height over the floor [m],
- z_0 – Is the terrain roughness factor.

$$\phi \cdot [L/L(z)] = 0{,}230 + 0{,}182\ \ln\,[L/L(z)]$$

(3.2.2.26)

Where:

$$0 \leq \phi \cdot \left[\frac{L}{L(z)}\right] \leq 1$$

(3.2.2.27)

- L is the span of the bridge [m], is the same one as the length of the bridge,
- $L(z)$ is the length of the turbulence [m], defined in chapter B.1.

The length of turbulence, L(z), can be calculated following Annex B.1 as:

$$L(z) = L_t \cdot \left(\frac{z}{z_t}\right)^{\propto} \qquad when \quad z \geq z_{min}$$

(3.2.2.28)

$$L(z) = L(z_{\min}) \qquad when \quad z \geq z_{min}$$

(3.2.2.29)

$$\propto = 0{,}67 + 0{,}05 \ln{(z_0)}$$

(3.2.2.30)

$$z_t = 200\, m \quad L_t = 300\, m$$

(3.2.2.31)

Due to $z = 6\, m$:

$$\propto = 0{,}67 + 0{,}05 \ln(0{,}05) = 0{,}52$$

(3.2.2.32)

$$L(z) = 300\, m \cdot \left(\frac{6}{200\, m}\right)^{0{,}52} = 48{,}44\, m$$

(3.2.2.33)

With this value we get:

$$\phi \cdot [L/L(z)] = 0{,}230 + 0{,}182 \ \ln{[L/L(z)]}$$

(3.2.2.34)

$$\phi \cdot [L/L(z)] = 0{,}230 + 0{,}182 \ \ln [20{,}4\ m/48{,}44\ m] \tag{3.2.2.35}$$

$$\phi \cdot [L/L(z)] = 0{,}0726 \tag{3.2.2.36}$$

Finally:

$$1 - \left(\frac{7}{C_o \cdot \ln\left(\frac{z}{z_0}\right) + 7}\right) \cdot \phi \cdot [L/L(z)] \tag{3.2.2.37}$$

$$1 - \left(\frac{7}{1{,}0 \cdot \ln\left(\frac{6}{0{,}05}\right) + 7}\right) \cdot 0{,}0726 = 0{,}956 \tag{3.2.2.38}$$

The value of $q_{w,y}$ will be:

$$q_{w,y} = 0{,}25 \cdot 0{,}956 \cdot F_{w,x} = 0{,}57\ KN/m \tag{3.2.2.39}$$

3.2.3 WEIGHT OF USE

According to EUROCODE UNE-EN 1991-1-1:2019//EN 1991-1-1:2002 table 6.1 defines categories of use, in this case C5 is selected.

Table 6.1 - Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores
[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

Image 17: Table 6.1 Categories of use (UNE-EN 1991-1-1:2019//EN 1991-1-1:2002)

In the same standard, table 6.2 gives us the values of imposed load.

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m²]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

Image 18: Table 6.2 Imposed loads on floors, balconies and stairs in buildings (UNE-EN 1991-1-1:2019//EN 1991-1-1:2002)

The value of continuous weight can be calculated by multiplying the value q_k and the width of the walking surface area and divided by the number of main beams.

$$q_k = \frac{5{,}0 \frac{KN}{m} \cdot 2{,}6\ m}{2} = 6{,}5\ \frac{KN}{m} \tag{3.2.3.1}$$

And the punctual charge:

$$Q_k = 4{,}5\ KN \tag{3.2.3.2}$$

3.3 LOAD COMBINATION

$$E_d = \sum_{j\geq 1} \gamma_{g,j} \cdot G_{k,j} + \gamma_{Ql} \cdot Q_{kl} + \sum_{i\geq 2} \gamma_{Qi} \cdot \Psi_{0i} \cdot Q_{ki} \tag{3.3.1}$$

E_d - Project Load

$\gamma_{g,j}$ – 1,35 – Coefficient of safety for continuous load

γ_{QL} – 1.5 – Variable load safety coefficient

Ψ_{0i} – Reduction coefficient

G_{KJ} – Permanent load

Q_{KL} –Variable load

Despite software Autodesk Robot calculates automatically combination according to the standards, it is always combinient to manualy calculate them.

There are defined four possible combinations:

$$ULS1 = 1.35 \cdot SW + 1.5 \cdot U + 1.5 \cdot 0.5 \cdot SN + 1.5 \cdot 0.6 \cdot W2$$
$$ULS2 = 1.35 \cdot SW + 1.5 \cdot 0.7 \cdot U + 1.5 \cdot SN + 1.5 \cdot 0.6 \cdot W2$$
$$ULS3 = 1.35 \cdot SW + 1.5 \cdot 0.7 \cdot U + 1.5 \cdot 0.5 \cdot SN + 1.5 \cdot W2$$
$$ULS4 = 1 \cdot SW + 1.5 \cdot W1 \tag{3.3.2}$$

Where:

- SW – Shelf Weight,
- U- Use,
- SN- Snow,
- W- Wind.

The continuous load over the two main beams is:

$$ULS1 = 1,35 \cdot 0,66 + 1,5 \cdot 6,5 + 1.5 \cdot 0.5 \cdot 0,16 + 1,5 \cdot 0,6 \cdot 0,25 = 11,98\ kN/m$$
$$ULS2 = 1,35 \cdot 0,66 + 1,5 \cdot 0,7 \cdot 6,5 + 1.5 \cdot 0,16 + 1,5 \cdot 0,6 \cdot 0,25 = 8,17\ kN/m$$
$$ULS3 = 1,35 \cdot 0,66 + 1,5 \cdot 0,7 \cdot 6,5 + 1.5 \cdot 0.5 \cdot 0,16 + 1,5 \cdot 0,25 = 8,21\ kN/m$$
$$ULS4 = 1 \cdot 0,66 + 1.5 \cdot (-0,25) = 0,285\ kN/m \tag{3.3.3}$$

3.4 STRUCTURAL ANALISY

Using the software Autodesk, the following results have been achieved:

UNE-EN 1993-1:2013/A1:2014 - Verificación de las barras (ELU) 1A26

Resultados Mensajes

Barra	Perfil	Material	Lay	Laz	Solicit.	Caso
13 Barra 13	HEA 100	S 275	64.11	103.58	0.00	9 ULS3
20 Barra 20	HEA 100	S 275	64.11	103.58	0.02	9 ULS3
4	HEB 120	S 275	129.70	214.00	0.07	10 ULS4
8 Barra 8	HEB 120	S 275	129.70	214.00	0.08	10 ULS4
24 Barra 24	HEB 120	S 275	174.41	287.76	0.11	7 ULS1
25	HEB 120	S 275	174.41	287.76	0.11	9 ULS3
10 Barra 10	HEA 100	S 275	64.11	103.58	0.14	9 ULS3
9 Barra 9	HEA 100	S 275	64.11	103.58	0.18	9 ULS3
18 Barra 18	HEA 100	S 275	43.15	69.72	0.28	7 ULS1
19 Barra 19	HEA 100	S 275	43.15	69.72	0.29	7 ULS1
2 Barra 2	HEA 200	S 275	101.43	168.64	0.36	10 ULS4
5 Barra 5	HEA 200	S 275	72.45	120.46	0.37	9 ULS3
3 Barra 3	HEA 200	S 275	72.45	120.46	0.39	7 ULS1
7 Barra 7	HEA 200	S 275	72.45	120.46	0.40	9 ULS3
1 Barra 1	HEA 200	S 275	72.45	120.46	0.40	7 ULS1
17 Barra 17	HEA 100	S 275	207.14	334.63	0.53	7 ULS1
12 Barra 12	HEA 100	S 275	154.12	248.98	0.54	7 ULS1
14 Barra 14	HEA 100	S 275	207.14	334.63	0.54	7 ULS1
11 Barra 11	HEA 100	S 275	154.12	248.98	0.55	7 ULS1
23 Barra 23	HEA 100	S 275	154.12	248.98	0.55	7 ULS1
21 Barra 21	HEA 100	S 275	154.12	248.98	0.56	7 ULS1
26	HEB 120	S 275	129.70	214.00	0.62	10 ULS4
6 Barra 6	HEA 200	S 275	101.43	168.64	0.70	9 ULS3
22 Barra 22	HEB 120	S 275	129.70	214.00	0.70	10 ULS4
15 Barra 15	HEA 100	S 275	43.15	69.72	0.71	7 ULS1
16 Barra 16	HEA 100	S 275	43.15	69.72	0.71	7 ULS1

Image 19: Table of Steel profiles ordered by solicitation

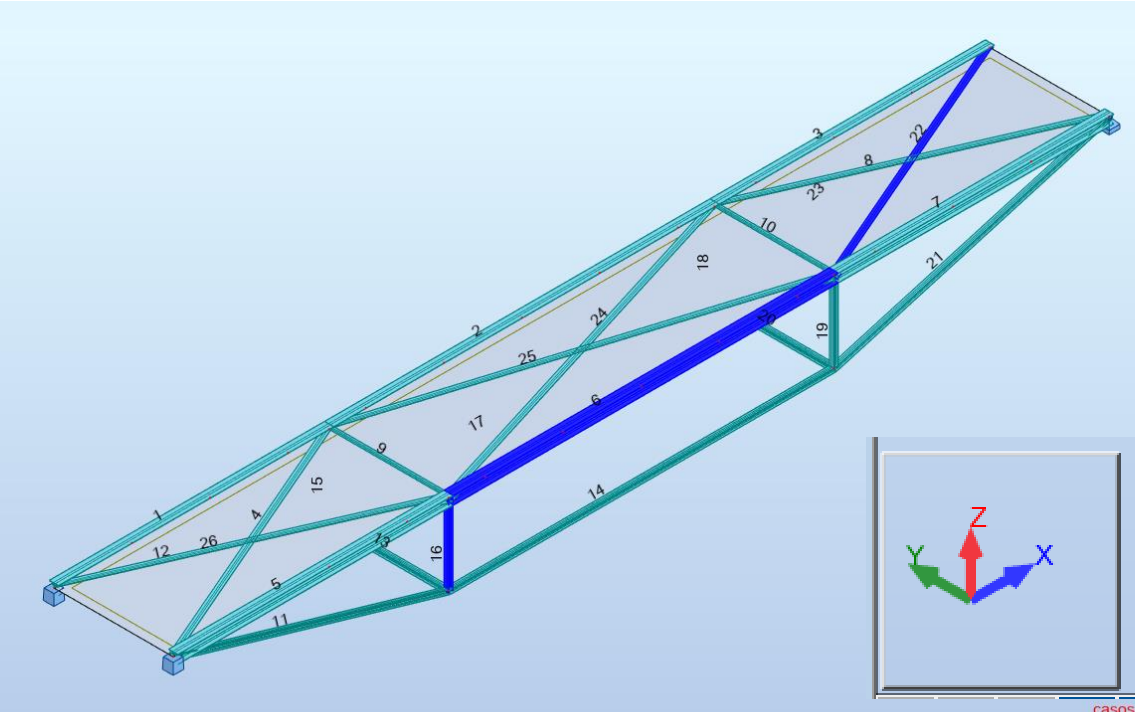


Image 20 of the mentioned steel profiles

The values of internal forces are:

Table 3.4.1: Internal forces

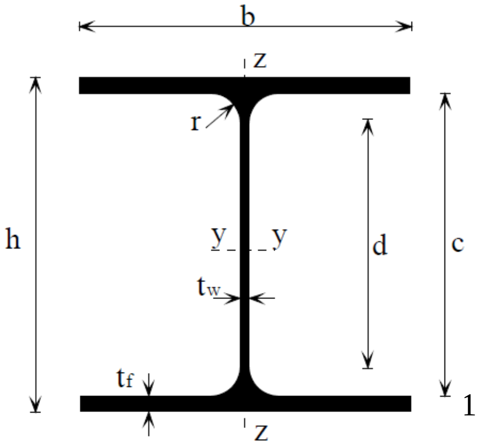
	N [kN]	$V_{y,Ed}$ [kN]	$V_{z,Ed}$ [kN]	M_y [kNm]	M_z [kNm]
HEB 120	71,85	-0,50	0,78	-0,40	-1,84
HEA 100	87,52	0,64	-8,70	4,40	0,28
HEA 200	20,60	-14,85	37,00	-46,44	-18,68

*According to the reference system of the upper image

4 DIMENSIONING

4.1 HEB 120 dimensioning

S275 $f_y = 275MPa$; $f_u = 430\ Mpa$; $\mathcal{E} = 0{,}92$



h = 120,0 mm	$W_{pl,y}$ = 165200 mm^3
b = 120,00 mm	$W_{pl,z}$ = 80970 mm^3
t_f = 11 mm	i_y = 50,4 mm
t_w = 6,5 mm	i_z = 30,6 mm
r = 12,0 mm	I_y = 864,4 cm^4
A = 3400 mm^2	I_z = 317,5 cm^4
c = 98,00 mm	I_t = 13,84 cm^4
E = 210000 N/mm^2	I_ω = 9410,0 cm^6
L = 6,54 m	$L_{u,yz}$ = 3,27 m

1.Cross-section classification:

a) Classification of web

$for\ \alpha > 0{,}5 \qquad c/t_w = 396\mathcal{E}/(13\,\alpha - 1)$
$for\ \alpha < 0{,}5 \qquad c/t_w = 36\mathcal{E}\,/\,\alpha$

(4.1.1)

$$N_{Ed} = 2 \cdot a \cdot tw \cdot fy$$

(4.1.2)

$$a = \frac{N_{Ed}}{2 \cdot tw \cdot fy}$$

(4.1.3)

$$a = \frac{71{,}85\ kN}{2 \cdot 6{,}5 \cdot 275\ N/mm^2} = \mathbf{20{,}09\ mm}$$

(4.1.4)

$$c = h - 2tf - 2r = 120mm - 2 \cdot 11mm - 2 \cdot 12mm = \mathbf{74\ mm}$$

(4.1.5)

$$\alpha \cdot c = c/2 + a \ \rightarrow \ \alpha = (c/2 + a)/c$$

(4.1.6)

$$\alpha = (74\ mm/2 + 20{,}09\ mm)/74\ mm = \mathbf{0{,}77}$$

(4.1.7)

$\alpha = 0'77 > 0'5 \rightarrow c/t_w = 74$ mm / 6,5 mm $= 11{,}38$
(4.1.8)

$$396\varepsilon/(13\alpha - 1) = (396 \cdot 0{,}92)/(13 \cdot 0'77 - 1) = 40{,}43 \quad (4.1.9)$$

$$11{,}38 \leq 40{,}43 \quad (4.1.10)$$

Web is in the 1st class

b) Classification of flange

$c/t_f < 9\varepsilon$

$c = b/2 - tw/2 - r$
$c = 120mm/2 - 6{,}5mm/2 - 12mm = \mathbf{44{,}75\ mm}$
(4.1.11)

$c/t_f = 44{,}75\ mm\ /\ 11\ mm = 4{,}06$
(4.1.12)

$4{,}06 < 9 \cdot 0{,}92$ flange is in the 1st class
(4.1.13)

Beam HEB 120 s275 is in the 1st class for compression.

2. Resistance to compression force:

$$N_{c,Rd} = N_{pl,Rd} = A \cdot f_y / \gamma_{M0} = 3400\text{mm}^2 \cdot \frac{275\ \text{N/mm}^2}{1} = 925\ kN \quad (4.1.14)$$

$$N_{Ed} < N_{c,Rd} \quad (4.1.15)$$

$71{,}85\ kN < 925\ kN$ the cross section can withstand the axial compressive force
(4.1.16)

3. Bending moment resistance:

· About y-y axis:

$$M_{pl,y,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = 165200\text{mm}^3 \cdot \frac{275\ \text{N/mm}^2}{1} = 45{,}43\ \text{kNm} \quad (4.1.17)$$

$$M_{pl,y,Rd} > M_{y,Ed} \tag{4.1.18}$$

$$45{,}43\ kNm > 0{,}4\ kNm \tag{4.1.19}$$

· About z-z axis:

$$M_{pl,z,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = 80970\mathrm{mm}^3 \cdot \frac{275\ \mathrm{N/mm}^2}{1} = 22{,}26\ kN \cdot m \tag{4.1.20}$$

$$M_{pl,z,Rd} > M_{z,Ed} \tag{4.1.21}$$

$$22{,}26\ kNm > 1{,}84\ kNm \tag{4.1.22}$$

The condition is fulfilled.

4. Resistance to axial forde and bending moment:

$$\frac{N_{Ed}}{N_{Ed}} + \frac{M_{y,Ed}}{M_{pl,y,Rd}} + \frac{M_{z,Ed}}{M_{pl,z,Rd}} \leq 1 \tag{4.1.23}$$

$$\frac{71{,}85\ kNm}{925\ kNm} + \frac{0{,}4\ kNm}{45{,}43\ kNm} + \frac{1{,}84\ kNm}{22{,}26\ kNm} = 0{,}169 \tag{4.1.24}$$

$$0{,}169 < 1 \tag{4.1.25}$$

The condition is fulfilled.

5. Resistance to shear force:

· $\mathcal{J}_m$ y direction:

$$V_{pl,y,Rd} > V_{y,Ed} \tag{4.1.26}$$

$$V_{pl,y,Rd} = A_{v,y} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} \tag{4.1.27}$$

$$A_{v,y} = A - \left(h - 2 \cdot t_f\right) \cdot t_w \tag{4.1.28}$$

$$A_{v,y} = 3400mm^2 - (120mm - 2 \cdot 11mm) \cdot 6{,}5mm = 2763mm^2 \tag{4.1.29}$$

$$V_{pl,y,Rd} = 2763mm^2 \cdot \frac{275\ \mathrm{N/mm^2}/\sqrt{3}}{1} = 438{,}68\ kN \tag{4.1.30}$$

$$438{,}68\ kN > 0{,}5\ kN \tag{4.1.31}$$

· $\mathcal{J}_m$ z direction :

$$V_{pl,z,Rd} > V_{z,Ed} \tag{4.1.32}$$

$$V_{pl,z,Rd} = A_{v,z} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} \tag{4.1.33}$$

$$A_{v,z} = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f \tag{4.1.34}$$

$$A_{v,z} = 3400mm^2 - 2 \cdot 120mm \cdot 11mm + (6{,}5mm + 2 \cdot 12mm) \cdot 11mm = 1095{,}5mm^2 \tag{4.1.35}$$

$$V_{pl,z,Rd} = 1095{,}5mm^2 \cdot \frac{275\ \mathrm{N/mm^2}/\sqrt{3}}{1} = 173{,}93\ kN \tag{4.1.36}$$

$$173{,}93\ kN > 0{,}78\ kN \tag{4.1.37}$$

6. Buckling resistance:

· About y-y axis:

$$\lambda_y = \frac{L_{u,y}}{i_y} = 3270mm/50{,}4mm = 64{,}88 \tag{4.1.38}$$

$$\lambda_1 = 93'9 \cdot \mathcal{E} = 93'9 \cdot 0{,}92 = 86{,}38 \tag{4.1.39}$$

$$\overline{\lambda_y} = \lambda_y/\lambda_1 = 64{,}88/86{,}38 = 0'751 \tag{4.1.40}$$

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Image 21: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

The selected curve is type b with a value of $\alpha = 0{,}34$

$$\Phi = 0{,}5 \cdot (1 + \alpha_y \cdot (\overline{\lambda_y} - 0{,}2) + \overline{\lambda_y}^2) \tag{4.1.41}$$

$$\Phi = 0{,}5 \cdot (1 + 0{,}34 \cdot (0'751 - 0{,}2) + 0'751^2) = 0{,}875 \tag{4.1.42}$$

$$\chi_y = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_y}^2}} \tag{4.1.43}$$

$$\chi_y = \frac{1}{0{,}875 + \sqrt{0{,}875^2 - 0'751^2}} = 0{,}754$$

(4.1.44)

· About z-z axis:

$$\lambda_z = \frac{L_{u,z}}{i_z} = 3270mm/30{,}6mm \ = \ 106{,}86$$

(4.1.45)

$$\lambda_1 = 93'9 \cdot \mathcal{E} = 93'9 \cdot 0{,}92 = 86{,}38$$

(4.1.46)

$$\overline{\lambda_z} = \ z/\lambda_1 = 106{,}86/86{,}38 = \ 1{,}237$$

(4.1.47)

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Image 22: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

The selected curve is the type c with a value of $\alpha = 0{,}49$

$$\Phi \ = \ 0{,}5 \cdot (1 + \alpha_z \cdot (\overline{\lambda_z} \ - \ 0{,}2) + \ \overline{\lambda_z}^{\,2})$$

(4.1.48)

$$\Phi \ = 0{,}5 \cdot (1 + 0{,}49 \cdot (1{,}237 - 0{,}2) + 1'237^2) = 1{,}54$$

(4.1.49)

$$\chi_z = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_z}^{\,2}}}$$

(4.1.50)

$$\chi_z = \frac{1}{1{,}54 + \sqrt{1{,}54^2 - 1{,}237^{-2}}} = 0{,}4059$$

(4.1.51)

· Lateral buckling:

$$M_{cr} = C_1 \cdot \frac{\pi}{L_u} \cdot \sqrt{E \cdot I_z \cdot G \cdot I_t} \cdot \sqrt{1 + \frac{\pi^2 \cdot E \cdot I_\omega}{L_u^2 \cdot G \cdot I_t}}$$

(4.1.52)

Where:

- M_{cr} – Elastic critical moment of lateral buckling,
- $C_1 = 1{,}13$,
- $E = 2{,}1 \cdot 10^5 \text{ N/mm}^2$,
- I_z – Moment of inertia around the weak axis,
- I_ω – Torsion moment of inertia,
- I_t – Torsion of the inertia moment at the uniform torsion,
- L_u –Buckling distance = 3270 mm,
- G – Shear Module of Steel.

$$M_{cr} = 1{,}13 \cdot \frac{\pi}{3270mm} \cdot \sqrt{2{,}1 \cdot 10^5 N/mm^2 \cdot 864{,}4 \cdot 10^4 mm^4 \cdot 80000 N/mm^2 \cdot 13{,}84 \cdot 10^4 mm^4} \cdot \sqrt{1 + \frac{\pi^2 \cdot 2{,}1 \cdot 10^5 N/mm^2 \cdot 9410 \cdot 10^6 mm^6}{3270^2 mm \cdot 80000 N/mm^2 \cdot 13{,}84 \cdot 10^4 mm^4}} = 166{,}1 \cdot 10^6\ Nmm$$

(4.1.52)

$$\lambda_{LT} = \sqrt{\frac{\omega_{pl,y} \cdot f_y}{M_{cr}}}$$

(4.1.53)

$$\lambda_{LT} = \sqrt{\frac{165200\, mm^3 \cdot 275N/mm^2}{166{,}1 \cdot 10^6 Nmm}} = 0{,}5229$$

(4.1.54)

Table 6.3: Recommended values for imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

Image 23: Table of recommended values of imperfection coefficient for lateral buckling (EN 1993-1-1:2005)

Table 6.4: Recommended values for lateral torsional buckling curves for cross-sections using equation (6.56)

Cross-section	Limits	Buckling curve
Rolled I-sections	h/b ≤ 2 h/b > 2	**a** **b**
Welded I-sections	h/b ≤ 2 h/b > 2	**c** **d**
Other cross-sections	-	**d**

Image 24: Table of recommended values for lateral torsional buckling curves for cross-sections (EN 1993-1-1:2005)

$$\frac{h}{b} = \frac{120mm}{120mm} = 1 < 2$$

(4.1.55)

The selected curve is a with a value of $\alpha = 0{,}21$

$$\Phi = 0{,}5 \cdot (1 + \alpha_{LT} \cdot (\overline{\lambda_{LT}} - 0{,}2) + \overline{\lambda_{LT}}^2)$$

(4.1.56)

$$\Phi = 0{,}5 \cdot (1 + 0{,}21 \cdot (0{,}5229 - 0{,}2) + 0{,}5229^2) = 0{,}67$$

(4.1.57)

$$\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_{LT}}^2}}$$

(4.1.58)

$$\chi_{\mathrm{LT}} = \frac{1}{0{,}67 + \sqrt{0{,}67^2 - 0{,}5229^2}} = 0{,}918$$

(4.1.59)

7. Profile resistance (6.3.3 EN 1993-1-1:2005)

$$\frac{N_{Ed}}{\frac{\chi_{\mathrm{y}} \cdot A \cdot f_y}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{\mathrm{LT}} \cdot \omega_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{z,Ed}}{\frac{\omega_{pl,z} \cdot f_y}{\gamma_{M1}}} \leq 1{,}0$$

(4.1.60)

$$\frac{N_{Ed}}{\frac{\chi_{\mathrm{y}} \cdot A \cdot f_y}{\gamma_{M1}}} + k_{zy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{\mathrm{LT}} \cdot \omega_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{zz} \cdot \frac{M_{z,Ed}}{\frac{\omega_{pl,z} \cdot f_y}{\gamma_{M1}}} \leq 1{,}0$$

(4.1.61)

- $N_{Ed}, M_{y,Ed}$ and $M_{z,Ed}$ - Design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively,
- χ_y and χ_z - Reduction factors due to flexural buckling,
- χ_{LT} - Reduction factor due to lateral torsional buckling,
- $k_{yy},\ k_{yz}, k_{zy}, k_{zz}$ - Interaction factors.

According to table B.3 (EN 1993-1-1:2005):

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

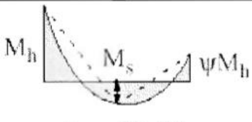
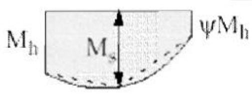
Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	C_{my} and C_{mz} and C_{mLT} concentrated load
	$-1 \leq \psi \leq 1$		$0{,}6 + 0{,}4\psi \geq 0{,}4$	
 $\alpha_s = M_s / M_h$	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0{,}2 + 0{,}8\alpha_s \geq 0{,}4$	$0{,}2 + 0{,}8\alpha_s \geq 0{,}4$
	$-1 \leq \alpha_s < 0$	$0 \leq \psi \leq 1$	$0{,}1 - 0{,}8\alpha_s \geq 0{,}4$	$-0{,}8\alpha_s \geq 0{,}4$
		$-1 \leq \psi < 0$	$0{,}1(1-\psi) - 0{,}8\alpha_s \geq 0{,}4$	$0{,}2(-\psi) - 0{,}8\alpha_s \geq 0{,}4$
 $\alpha_h = M_h / M_s$	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
	$-1 \leq \alpha_h < 0$	$0 \leq \psi \leq 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
		$-1 \leq \psi < 0$	$0{,}95 + 0{,}05\alpha_h(1+2\psi)$	AC2 $0{,}90 + 0{,}10\alpha_h(1+2\psi)$ AC2
For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0{,}9$ or AC2 C_{mz} AC2 $= 0{,}9$ respectively.				
C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:				
moment factor	bending axis	points braced in direction		
C_{my}	y-y	z-z		
C_{mz}	z-z	y-y		
C_{mLT}	y-y	y-y		

Image 25: Table of equivalent uniform moment factors Cm (EN 1993-1-1:2005)

The values of C_m are:

$$C_{my} = 0{,}95$$

$$C_{mz} = 0{,}95$$

$$C_{mLT} = 0{,}95$$

The iteration coefficients can be calculated following table B.2 (EN 1993-1-1:2005):

$$k_{yy} = C_{my} \cdot \left(1 + \left(\overline{\lambda_y} - 0{,}2\right) \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} \right) \leq C_{my} \cdot \left(1 + 0{,}8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} \right) \tag{4.1.62}$$

$$0{,}95 \cdot \left(1 + (0'751 - 0{,}2) \cdot \frac{71850\ N}{\frac{0{,}754 \cdot 3400\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}}\right) \leq 0{,}95 \cdot \left(1 + 0{,}8 \cdot \frac{71850\ N}{\frac{0{,}754 \cdot 3400\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}}\right)$$

(4.1.63)

$$\mathbf{1{,}003} \leq 1{,}027$$

$$k_{zz} = C_{mz} \cdot \left(1 + \left(2 \cdot \overline{\lambda_z} - 0{,}6\right) \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}}\right) \leq C_{mz} \cdot \left(1 + 1{,}4 \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}}\right)$$

(4.1.64)

$$0{,}95 \cdot \left(1 + (2 \cdot 1{,}237 - 0{,}6) \cdot \frac{71850\ N}{\frac{0{,}4059 \cdot 3400\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}}\right) \leq 0{,}95 \cdot \left(1 + 1{,}4 \cdot \frac{71850\ N}{\frac{0{,}4059 \cdot 3400\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}}\right)$$

(4.1.65)

$$1{,}287 \leq \mathbf{1{,}201}$$

$$k_{zy} = \left(1 - \frac{0{,}1 \cdot \overline{\lambda_z}}{(C_{mLT} - 0{,}25)} \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}}\right) \geq \left(1 - \frac{0{,}1}{(C_{mLT} - 0{,}25)} \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}}\right)$$

(4.1.66)

$$\left(1-\frac{0{,}1\cdot 1{,}237}{(0{,}95-0{,}25)}\cdot\frac{71850\,N}{\frac{0{,}4059\cdot 3400\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}\right)\geq\left(1-\frac{0{,}1}{(0{,}95-0{,}25)}\cdot\frac{71850\,N}{\frac{0{,}4059\cdot 3400\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}\right) \tag{4.1.67}$$

$$0{,}966\geq\mathbf{0,972}$$

$k_{yz}=$

$$k_{yz}=0{,}6\cdot k_{zz}=\mathbf{0,720} \tag{4.1.68}$$

Finally:

$k_{yy}=1{,}003$

$k_{zz}=1{,}201$

$k_{zy}=0{,}972$

$k_{yz}=0{,}720$

$$\frac{71850\,N}{\frac{0{,}754\cdot 3400\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}+1{,}003\cdot\frac{0{,}4\cdot 10^6\ N\cdot mm}{\frac{0{,}918\cdot 165200\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}720\cdot\frac{1{,}84\cdot 10^6\ N\cdot mm}{\frac{0{,}918\cdot 80970\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}=\mathbf{0,1763} \tag{4.1.69}$$

$$\mathbf{0,1763<1,0}$$

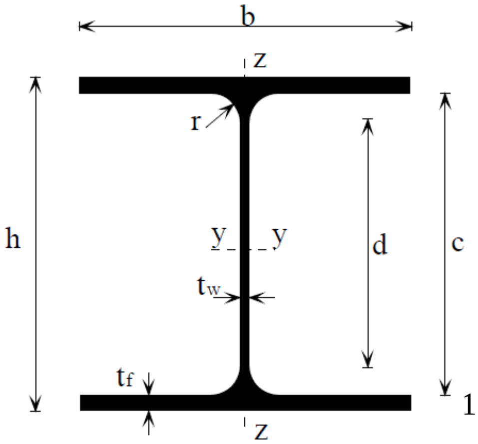
$$\frac{71850\,N}{\frac{0{,}754\cdot 3400\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}972\cdot\frac{0{,}4\cdot 10^6\ N\cdot mm}{\frac{0{,}918\cdot 165200\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}+1{,}201\cdot\frac{1{,}84\cdot 10^6\ N\cdot mm}{\frac{0{,}918\cdot 80970\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}=0{,}2193 \tag{4.1.70}$$

$$\mathbf{0,2193 < 1,0}$$

The resistance conditions are satisfied, the profile is OK.

4.2 HEA 100 dimensioning

S275 $f_y = 275MPa$; $f_u = 430\ Mpa$; $\mathcal{E} = 0{,}92$



h = 96,0 mm	$W_{pl,y}$ = 83010 mm^3
b = 100,00 mm	$W_{pl,z}$ = 41140 mm^3
t_f = 8 mm	i_y = 40,6 mm
t_w = 5 mm	i_z = 25,1 mm
r = 12,0 mm	I_y = 349,2 cm^4
A = 2120 mm^2	I_z = 133,8 cm^4
c = 80,00 mm	I_t = 5,24 cm^4
E = 210000 N/mm^2	I_ω = 2580 cm^6
L = 1,75 m	$L_{u,yz}$ = 1,75 m

1.Cross-section classification:

a) Classification of web

$for\ \alpha > 0{,}5 \qquad c/t_w = 396\mathcal{E}/(13\,\alpha - 1)$
$for\ \alpha < 0{,}5 \qquad c/t_w = 36\mathcal{E}\,/\,\alpha$

$$N_{Ed} = 2 \cdot a \cdot tw \cdot fy \tag{4.2.1}$$

$$a = \frac{N_{Ed}}{2 \cdot tw \cdot fy} \tag{4.2.2}$$

$$a = \frac{87{,}52\ kN}{2 \cdot 5 \cdot 275\ N/mm^2} = \mathbf{31{,}82\ mm} \tag{4.2.3}$$

$$c = h - 2tf - 2r = 96mm - 2 \cdot 8mm - 2 \cdot 12mm = \mathbf{56\ mm} \tag{4.2.4}$$

$$\alpha \cdot c = c/2 + a \ \rightarrow\ \alpha = (c/2 + a)/c$$

$$\alpha = (56\ mm/2 + 31{,}82\ mm)/56\ mm = \mathbf{1{,}068} \tag{4.2.5}$$

$$\alpha = 1{,}06 > 0'5 \ \rightarrow c/t_w = 56\text{ mm} / 5\text{ mm} = 11{,}2 \tag{4.2.6}$$

$$396\mathcal{E}/(13\alpha - 1) = (396 \cdot 0{,}92)/(13 \cdot 1{,}06 - 1) = 28{,}50 \tag{4.2.7}$$

$$11{,}2 \leq 28{,}50 \tag{4.2.8}$$

Web is in the 1st class

b) Classification of flange

$c/t_f < 9\mathcal{E}$ $\quad c = b/2 - tw/2 - r$

$$c = 100mm/2 - 5mm/2 - 12mm = \mathbf{35{,}5\ mm} \tag{4.2.9}$$

$$c/t_f = 35{,}5\ mm\ /\ 8\ mm = 4{,}43 \tag{4.2.10}$$

$4{,}43 < 9 \cdot 0{,}92$ flange is in the 1st class (4.2.11)

Beam HEA 100 S275 is in the 1st class for compression.

2. Resistance to compression force:

$$N_{c,Rd} = N_{pl,Rd} = A \cdot f_y / \gamma_{M0} = 2120\text{mm}^2 \cdot \frac{275\ \text{N/mm}^2}{1} = 583\ kN \tag{4.2.12}$$

$$N_{Ed} < N_{c,Rd} \tag{4.2.13}$$

$87{,}52\ kN < 583\ kN$ the cross section can withstand the axial compressive force (4.2.14)

3. Bending moment resistance:

· About y-y axis:

$$M_{pl,y,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = 83010\text{mm}^3 \cdot \frac{275\ \text{N/mm}^2}{1} = 22{,}82\ \text{kNm} \tag{4.2.15}$$

$$M_{pl,y,Rd} > M_{y,Ed}$$

(4.2.16)

$$22{,}82\ kN \cdot m > 4{,}4\ kN \cdot m$$

(4.2.17)

· About z-z axis:

$$M_{pl,z,Rd} \ = \ W_{pl,y} \cdot f_y \ /\gamma_{M0} \ = 41140\text{mm}^3 \cdot \ \frac{275\ \text{N/mm}^2}{1} \ = \ 11{,}31\ kNm$$

(4.2.18)

$$M_{pl,z,Rd} > M_{z,Ed}$$

(4.2.19)

$$11{,}31\ kNm > 0{,}28\ kNm$$

(4.2.20)

The condition is fulfilled.

4. Resistance to axial forde and bending moment:

$$\frac{N_{Ed}}{N_{Ed}} + \frac{M_{y,Ed}}{M_{pl,y,Rd}} + \frac{M_{z,Ed}}{M_{pl,z,Rd}} \leq 1$$

(4.2.21)

$$\frac{87{,}52\ kNm}{583\ kNm} + \frac{4{,}4\ kNm}{22{,}82\ kNm} + \frac{0{,}28\ kNm}{11{,}31\ kNm} = 0{,}367$$

(4.2.22)

$$0{,}367 < 1$$

(4.2.23)

The condition is fulfilled.

5. Resistance to shear force:

· $\mathcal{J}_m$ y direction :

$$V_{pl,y,Rd} > V_{y,Ed}$$

(4.2.24)

$$V_{pl,y,Rd} = A_{v,y} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

(4.2.25)

$$A_{v,y} = A - \left(h - 2 \cdot t_f\right) \cdot t_w$$

(4.2.26)

$$A_{v,y} = 2120mm^2 - (96mm - 2 \cdot 8mm) \cdot 5mm = 1720mm^2$$

(4.2.27)

$$V_{pl,y,Rd} = 1720mm^2 \cdot \frac{275\ \mathrm{N/mm^2}/\sqrt{3}}{1} = 273{,}08\ kN$$

(4.2.28)

$$273{,}08\ kN > 0{,}64\ kN$$

(4.2.29)

· $\mathcal{J}_m$ z direction :

$$V_{pl,z,Rd} > V_{z,Ed}$$

(4.2.30)

$$V_{pl,z,Rd} = A_{v,z} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

(4.2.31)

$$A_{v,z} = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f$$

(4.2.32)

$$A_{v,z} = 2120mm^2 - 2 \cdot 100mm \cdot 8mm + (5mm + 2 \cdot 12mm) \cdot 8mm = 752mm^2$$

(4.2.33)

$$V_{pl,z,Rd} = 752mm^2 \cdot \frac{275\ \mathrm{N/mm^2}/\sqrt{3}}{1} = 119{,}39\ kN$$

(4.2.34)

$$119{,}39\ kN > 8{,}70\ kN$$

(4.2.35)

6. Buckling resistance:

· About y-y axis:

$$\lambda_y = \frac{L_{u,y}}{i_y} = 1750mm/40{,}6mm = 43{,}1$$

(4.2.36)

$$\lambda_1 = 93'9 \cdot \mathcal{E} = 93'9 \cdot 0{,}92 = 86{,}38$$

(4.2.37)

$$\overline{\lambda_y} = \lambda_y/\lambda_1 = 43{,}1/86{,}38 = 0'4989$$

(4.2.38)

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Image 26: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

The selected curve is type b with a value of $\alpha = 0{,}34$

$$\Phi \ = \ 0{,}5 \cdot (1 + \alpha_y \cdot (\overline{\lambda_y} \ - \ 0{,}2) + \ \overline{\lambda_y}^{\,-2})$$

(4.2.39)

$$\Phi \ = 0{,}5 \cdot (1 + 0{,}34 \cdot (0'4989 - 0{,}2) + 0'4989^2) = 0{,}675$$

(4.2.40)

$$\chi_y = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_y}^{\,2}}}$$

(4.2.41)

$$\chi_y = \frac{1}{0{,}675 + \sqrt{0{,}675^2 - 0'4989^2}} = 0{,}8847$$

(4.2.42)

· About z-z axis:

$$\lambda_z = \frac{L_{u,z}}{i_z} = 1750mm/25{,}1mm \ = \ 69{,}72$$

(4.2.43)

$$\lambda_1 = 93'9 \cdot \mathcal{E} = 93'9 \cdot 0{,}92 = 86{,}38$$

(4.2.44)

$$\overline{\lambda_z} = \ z/\lambda_1 = 69{,}72/86{,}38 = \ 0{,}807$$

(4.2.45)

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Image 27: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

The selected curve is type c with a value of $\alpha = 0{,}49$

$$\Phi = 0{,}5 \cdot (1 + \alpha_z \cdot (\overline{\lambda_z} - 0{,}2) + \overline{\lambda_z}^2) \tag{4.2.46}$$

$$\Phi = 0{,}5 \cdot (1 + 0{,}49 \cdot (0{,}807 - 0{,}2) + 0{,}807^2) = 0{,}974 \tag{4.2.47}$$

$$\chi_z = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_z}^2}} \tag{4.2.48}$$

$$\chi_z = \frac{1}{0{,}974 + \sqrt{0{,}974^2 - 0{,}807^2}} = 0{,}6579 \tag{4.2.49}$$

· Lateral buckling:

$$M_{cr} = C_1 \cdot \frac{\pi}{L_u} \cdot \sqrt{E \cdot I_z \cdot G \cdot I_t} \cdot \sqrt{1 + \frac{\pi^2 \cdot E \cdot I_\omega}{L_u^2 \cdot G \cdot I_t}} \tag{4.2.50}$$

Where:

- M_{cr} – Elastic critical moment of lateral buckling,
- $C_1 = 1{,}13$,
- $E = 2{,}1 \cdot 10^5\ \mathrm{N/mm^2}$,
- I_z – Moment of inertia around the weak axis,
- I_ω – Torsion moment of inertia,
- I_t – Torsion of the inertia moment at the uniform torsion,
- L_u –Buckling distance = 3270 mm,
- G – Shear Module of Steel.

$$M_{cr} = 1{,}13 \cdot \frac{\pi}{1750mm} \cdot \sqrt{2{,}1 \cdot 10^5 N/mm^2 \cdot 133{,}8 \cdot 10^4 mm^4 \cdot 80000 N/mm^2 \cdot 5{,}24 \cdot 10^4 mm^4} \cdot \sqrt{1 + \frac{\pi^2 \cdot 2{,}1 \cdot 10^5 N/mm^2 \cdot 2580 \cdot 10^6 mm^6}{1750^2 mm \cdot 80000 N/mm^2 \cdot 5{,}24 \cdot 10^4 mm^4}} = 82{,}86 \cdot 10^6\ Nmm$$

(4.2.51)

$$\lambda_{LT} = \sqrt{\frac{\omega_{pl,y} \cdot f_y}{M_{cr}}}$$

(4.2.52)

$$\lambda_{LT} = \sqrt{\frac{83010\ mm^3 \cdot 275 N/mm^2}{82{,}86 \cdot 10^6 Nmm}} = 0{,}5248$$

(4.2.53)

Table 6.3: Recommended values for imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

Image 28: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

Table 6.4: Recommended values for lateral torsional buckling curves for cross-sections using equation (6.56)

Cross-section	Limits	Buckling curve
Rolled I-sections	h/b ≤ 2 h/b > 2	**a** **b**
Welded I-sections	h/b ≤ 2 h/b > 2	**c** **d**
Other cross-sections	-	**d**

Image 29: Table of recommended values for lateral torsional buckling curves for cross-sections (EN 1993-1-1:2005)

$$\frac{h}{b} = \frac{96mm}{100mm} = 0{,}96 < 2$$

(4.2.54)

The selected curve is a with a value of $\alpha = 0{,}21$

$$\Phi = 0{,}5 \cdot (1 + \alpha_{LT} \cdot (\overline{\lambda_{LT}} - 0{,}2) + \overline{\lambda_{LT}}^2)$$

(4.2.55)

$$\Phi = 0{,}5 \cdot (1 + 0{,}21 \cdot (0{,}5248 - 0{,}2) + 0{,}5248^2) = 0{,}6718$$

(4.2.56)

$$\chi_{\mathrm{LT}} = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_{LT}}^{\,2}}}$$

(4.2.57)

$$\chi_{\mathrm{LT}} = \frac{1}{0{,}6718 + \sqrt{0{,}6718^2 - 0{,}5248^2}} = 0{,}9163$$

(4.2.58)

7. Profile resistance (6.3.3 EN 1993-1-1:2005)

$$\frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{LT} \cdot \omega_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{z,Ed}}{\frac{\omega_{pl,z} \cdot f_y}{\gamma_{M1}}} \leq 1{,}0 \tag{4.2.59}$$

$$\frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} + k_{zy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{LT} \cdot \omega_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{zz} \cdot \frac{M_{z,Ed}}{\frac{\omega_{pl,z} \cdot f_y}{\gamma_{M1}}} \leq 1{,}0 \tag{4.2.60}$$

- $N_{Ed}, M_{y,Ed}$ and $M_{z,Ed}$ - Design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively,
- χ_y and χ_z - Reduction factors due to flexural buckling from 6.3.1,
- χ_{LT} - Reduction factor due to lateral torsional buckling from 6.3.2,
- $k_{yy},\ k_{yz}, k_{zy}, k_{zz}$ - Interaction factors.

According to table B.3 (EN 1993-1-1:2005):

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	concentrated load
M, ψM	$-1 \leq \psi \leq 1$		$0{,}6 + 0{,}4\psi \geq 0{,}4$	
M_h, M_s, ψM_h; $\alpha_s = M_s/M_h$	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0{,}2 + 0{,}8\alpha_s \geq 0{,}4$	$0{,}2 + 0{,}8\alpha_s \geq 0{,}4$
	$-1 \leq \alpha_s < 0$	$0 \leq \psi \leq 1$	$0{,}1 - 0{,}8\alpha_s \geq 0{,}4$	$-0{,}8\alpha_s \geq 0{,}4$
		$-1 \leq \psi < 0$	$0{,}1(1-\psi) - 0{,}8\alpha_s \geq 0{,}4$	$0{,}2(-\psi) - 0{,}8\alpha_s \geq 0{,}4$
M_h, M_s, ψM_h; $\alpha_h = M_h/M_s$	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
	$-1 \leq \alpha_h < 0$	$0 \leq \psi \leq 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
		$-1 \leq \psi < 0$	$0{,}95 + 0{,}05\alpha_h(1+2\psi)$	AC2 $0{,}90 + 0{,}10\alpha_h(1+2\psi)$ AC2

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0{,}9$ or AC2 C_{mz} AC2 = 0,9 respectively.

C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:

moment factor	bending axis	points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

Image 30: Table of equivalent uniform moment factors Cm (EN 1993-1-1:2005)

The values of C_m are:

$C_{my} = 0,95$

$C_{mz} = 0,95$

$C_{mLT} = 0,95$

The iteration coefficients can be calculated following table B.2 (EN 1993-1-1:2005):

$$k_{yy} = C_{my} \cdot \left(1 + \left(\overline{\lambda_y} - 0,2\right) \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} \right) \le C_{my} \cdot \left(1 + 0,8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} \right) \tag{4.2.61}$$

$$0,95 \cdot \left(1 + (0'4989 - 0,2) \cdot \frac{87520\ N}{\frac{0,8847 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \le 0,95 \cdot \left(1 + 0,8 \cdot \frac{87520\ N}{\frac{0,8847 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \tag{4.2.62}$$

$$\mathbf{0,998} \le 1,078$$

$$k_{zz} = C_{mz} \cdot \left(1 + \left(2 \cdot \overline{\lambda_z} - 0,6\right) \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}} \right) \le C_{mz} \cdot \left(1 + 1,4 \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}} \right) \tag{4.2.63}$$

$$0,95 \cdot \left(1 + (2 \cdot 0,807 - 0,6) \cdot \frac{87520\ N}{\frac{0,6579 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \le 0,95 \cdot \left(1 + 1,4 \cdot \frac{87520\ N}{\frac{0,6579 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \tag{4.2.64}$$

$$\mathbf{1,169} \le 1,253$$

$k_{zy} =$

$$\left(1 - \frac{0,1 \cdot \overline{\lambda_z}}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}}\right) \geq \left(1 - \frac{0,1}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}}\right)$$

(4.2.65)

$$\left(1 - \frac{0,1 \cdot 0,807}{(0,95 - 0,25)} \cdot \frac{87520\ N}{\frac{0,6579 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}}\right) \geq \left(1 - \frac{0,1}{(0,95 - 0,25)} \cdot \frac{87520\ N}{\frac{0,6579 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}}\right)$$

(4.2.66)

$$\mathbf{0,973} \geq 0,967$$

$k_{yz} =$

$$k_{yz} = 0,6 \cdot k_{zz} = \mathbf{0,701}$$

Finally:

$k_{yy} = 0,998$

$k_{zz} = 1,169$

$k_{zy} = 0,973$

$k_{yz} = 0,701$

$$\frac{87520\ N}{\frac{0,8847 \cdot 2120\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} + 0,998 \cdot \frac{4,4 \cdot 10^6\ N \cdot mm}{\frac{0,9163 \cdot 83010\text{mm}^3 \cdot 275\ \text{N/mm}^2}{1}} + 0,701 \cdot \frac{0,28 \cdot 10^6\ N \cdot mm}{\frac{0,9163 \cdot 41140\text{mm}^3 \cdot 275\ \text{N/mm}^2}{1}} = \mathbf{0,3985}$$

(4.2.67)

$$\mathbf{0,3985 < 1,0}$$

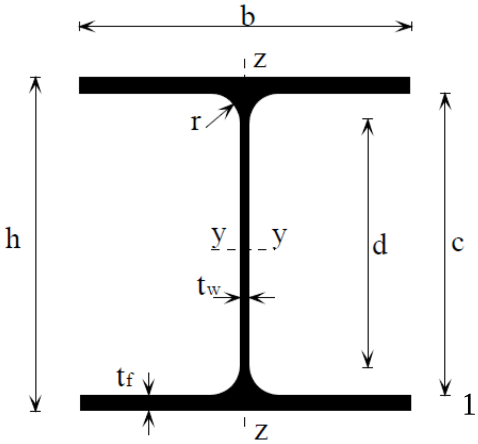
$$\frac{87520\,N}{\frac{0{,}8847\cdot 2120\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}973\cdot\frac{4{,}4\cdot 10^6\ N\cdot mm}{\frac{0{,}2668\cdot 83010\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}+1{,}169\cdot\frac{0{,}28\cdot 10^6\ N\cdot mm}{\frac{0{,}2668\cdot 41140\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}=\mathbf{0{,}4059} \tag{4.2.68}$$

$$\mathbf{0{,}4059 < 1{,}0}$$

The resistance conditions are satisfied, the profile is OK.

4.3 HEA 200 dimensioning

S275 $f_y = 275MPa$; $f_u = 430\ Mpa$; $\mathcal{E} = 0{,}92$



h = 190,0 mm	$W_{pl,y}$ = 429500 mm^3
b = 200,00 mm	$W_{pl,z}$ = 203800 mm^3
t_f = 10 mm	i_y = 82,8 mm
t_w = 6,5 mm	i_z = 49,8 mm
r = 18,0 mm	I_y = 3692 cm^4
A = 5380 mm^2	I_z = 1336 cm^4
c = 134,00 mm	I_t = 20,98 cm^4
E = 210000 N/mm^2	I_ω = 108000 cm^6
L = 8,4 m	$L_{u,yz}$ = 8,4 m

1.Cross-section classification:

a) Classification of web

$for\ \alpha > 0{,}5 \qquad c/t_w = 396\mathcal{E}/(13\,\alpha - 1)$
$for\ \alpha < 0{,}5 \qquad c/t_w = 36\mathcal{E}/\alpha$

(4.3.1)

$$N_{Ed} = 2 \cdot a \cdot tw \cdot fy$$

$$a = \frac{N_{Ed}}{2 \cdot tw \cdot fy}$$

(4.3.2)

$$a = \frac{20{,}6\ kN}{2 \cdot 6{,}5 \cdot 275\ N/mm^2} = \mathbf{5{,}76\ mm}$$

(4.3.3)

$$c = h - 2tf - 2r = 190mm - 2 \cdot 10mm - 2 \cdot 18mm = \mathbf{134\ mm}$$

(4.3.4)

$$\alpha \cdot c = c/2 + a \ \rightarrow\ \alpha = (c/2 + a)/c$$

(4.3.5)

$$\alpha = (134\ mm/2 + 5{,}76\ mm)/134\ mm = \mathbf{0{,}543}$$

(4.3.6)

$\alpha = 0{,}543 > 0'5 \rightarrow c/t_w = 134\text{ mm} / 6{,}5\text{ mm} = 20{,}61$

(4.3.7)

$$396\mathcal{E}/(13\alpha - 1) = (396 \cdot 0{,}92)/(13 \cdot 0{,}543 - 1) = 60{,}12$$

(4.3.8)

$$20{,}61 \leq 60{,}12$$

(4.3.9)

Web is in the 1st class

b) Classification of flange

$c/t_f < 9\varepsilon$ $\quad\quad\quad\quad c = b/2 - tw/2 - r$

(4.3.10)

$$c = \frac{200mm}{2} - \frac{6{,}5mm}{2} - 18mm = \mathbf{78{,}75\ mm}$$

(4.3.11)

$c/t_f = 78{,}75\ mm\ /\ 10\ mm\ =\ 7{,}875$

(4.3.12)

$7{,}875 < 9 \cdot 0{,}92$ flange is in the 1st class

(4.3.13)

Beam HEA 200 S275 is in the 1st class for compression.

2. Resistance to compression force:

$$N_{c,Rd} = N_{pl,Rd} = A \cdot f_y / \gamma_{M0} = 5830\text{mm}^2 \cdot \frac{275\ \text{N/mm}^2}{1} = 1603{,}25\ kN$$

(4.3.14)

$$N_{Ed} < N_{c,Rd}$$

(4.3.15)

$20{,}60\ kN < 1603{,}25\ kN$ the cross section can withstand the axial compressive force

(4.3.16)

3. Bending moment resistance:

· About y-y axis:

$$M_{pl,y,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = 429500\text{mm}^3 \cdot \frac{275\ \text{N/mm}^2}{1} = 118{,}11\ \text{kNm}$$

(4.3.17)

$$M_{pl,y,Rd} > M_{y,Ed}$$

(4.3.18)

$$118{,}11\ kN \cdot m > 46{,}44\ kN \cdot m$$

(4.3.19)

· About z-z axis:

$$M_{pl,z,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = 203800\text{mm}^3 \cdot \frac{275\ \text{N/mm}^2}{1} = 56{,}05\ kNm$$

(4.3.20)

$$M_{pl,z,Rd} > M_{z,Ed} \tag{4.3.21}$$

$$56{,}05\ kNm > 18{,}68\ kNm \tag{4.3.22}$$

The condition is fulfilled.

4. Resistance to axial forde and bending moment:

$$\frac{N_{Ed}}{N_{Ed}} + \frac{M_{y,Ed}}{M_{pl,y,Rd}} + \frac{M_{z,Ed}}{M_{pl,z,Rd}} \leq 1 \tag{4.3.23}$$

$$\frac{20{,}60\ kNm}{1603{,}25\ kNm} + \frac{46{,}44\ kNm}{118{,}11\ kNm} + \frac{18{,}68\ kNm}{56{,}05\ kNm} = 0{,}739 \tag{4.3.24}$$

$$0{,}739 < 1 \tag{4.3.25}$$

The condition is fulfilled.

5. Resistance to shear force:

· $\mathcal{J}_m$ y direction :

$$V_{pl,y,Rd} > V_{y,Ed} \tag{4.3.26}$$

$$V_{pl,y,Rd} = A_{v,y} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} \tag{4.3.27}$$

$$A_{v,y} = A - \left(h - 2 \cdot t_f\right) \cdot t_w \tag{4.3.28}$$

$$A_{v,y} = 5380mm^2 - (190mm - 2 \cdot 10mm) \cdot 6{,}5mm = 4275mm^2 \tag{4.3.29}$$

$$V_{pl,y,Rd} = 4275mm^2 \cdot \frac{275\ \mathrm{N/mm^2}/\sqrt{3}}{1} = 678{,}74\ kN \tag{4.3.30}$$

$$678{,}74\ kN > 14{,}85\ kN \tag{4.3.31}$$

· $\mathcal{J}_m$ z direction :

$$V_{pl,z,Rd} > V_{z,Ed}$$

(4.3.32)

$$V_{pl,z,Rd} = A_{v,z} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

(4.3.33)

$$A_{v,z} = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f$$

(4.3.34)

$$A_{v,z} = 5380mm^2 - 2 \cdot 200mm \cdot 10mm + (6{,}5mm + 2 \cdot 18mm) \cdot 10mm = 1805mm^2$$

(4.3.35)

$$V_{pl,z,Rd} = 1805mm^2 \cdot \frac{275 \text{ N/mm}^2/\sqrt{3}}{1} = 286{,}58\ kN$$

(4.3.36)

$$286{,}58\ kN > 37{,}0\ kN$$

(4.3.37)

6. Buckling resistance:

· About y-y axis:

$$\lambda_y = \frac{L_{u,y}}{i_y} = 8400mm/82{,}8mm \ = \ 101{,}44$$

(4.3.38)

$$\lambda_1 = 93'9 \cdot \mathcal{E} = 93'9 \cdot 0{,}92 = 86{,}38$$

(4.3.39)

$$\overline{\lambda_y} = \ \lambda_y/\lambda_1 = 101{,}44/86{,}38 = \ 1{,}174$$

(4.3.40)

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Image 31: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

The selected curve is the type b with a value of $\alpha = 0{,}34$

$$\Phi \ = \ 0{,}5 \cdot (1 + \alpha_y \cdot (\overline{\lambda_y} \ - \ 0{,}2) + \ \overline{\lambda_y}^{-2}) \tag{4.3.41}$$

$$\Phi \ = 0{,}5 \cdot (1 + 0{,}34 \cdot (1{,}174 - 0{,}2) + 1{,}174^2) = 1{,}354 \tag{4.3.42}$$

$$\chi_y = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_y}^2}} \tag{4.3.43}$$

$$\chi_y = \frac{1}{1{,}354 + \sqrt{1{,}354^2 - 1{,}174^2}} = 0{,}4925 \tag{4.3.44}$$

· About z-z axis:

$$\lambda_z = \frac{L_{u,z}}{i_z} = 8400mm/49{,}8mm \ = \ 168{,}67 \tag{4.3.45}$$

$$\lambda_1 = 93'9 \cdot \mathcal{E} = 93'9 \cdot 0{,}92 = 86{,}38 \tag{4.3.46}$$

$$\overline{\lambda_z} = \ z/\lambda_1 = 168{,}67/86{,}38 = \ 1{,}952 \tag{4.3.47}$$

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Image 32: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005)

The selected curve is type c with a value of $\alpha = 0{,}49$

$$\Phi \ = \ 0{,}5 \cdot (1 + \alpha_z \cdot (\overline{\lambda_z} \ - \ 0{,}2) + \ \overline{\lambda_z}^{2}) \tag{4.3.48}$$

$$\Phi \ = 0{,}5 \cdot (1 + 0{,}49 \cdot (1{,}952 - 0{,}2) + 1{,}952^2) = 2{,}834 \tag{4.3.49}$$

$$\chi_z = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_z}^2}}$$

(4.3.50)

$$\chi_z = \frac{1}{2{,}834 + \sqrt{2{,}834^2 - 1{,}952^2}} = 0{,}2045$$

(4.3.51)

· Lateral buckling:

$$M_{cr} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{(k \cdot L)^2} \cdot \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \cdot \frac{I_\omega}{I_z} + \frac{(k \cdot L)^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z} + \left(C_z \cdot z_g\right)^2} - C_2 \cdot z_g\right]$$

(4.3.52)

Where:

- M_{cr} – Elastic critical moment of lateral buckling,
- $C_1 = 1{,}285 \qquad C_2 = 1{,}562$,
- $E = 2{,}1 \cdot 10^5\ \mathrm{N/mm^2}$,
- I_z – Moment of inertia around the weak axis,
- I_ω – Torsion moment of inertia,
- I_t – Torsion of the inertia moment at the uniform torsion,
- L_u –Buckling distance = 8400 mm,
- G – Shear Module of Steel,
- $k = k_w = 1$
- $z_g = h/2 = 190\ mm/2 = 95\ mm$ (HEA 200)

$$M_{cr} = 1{,}285 \cdot \frac{\pi^2 \cdot 2{,}1 \cdot 10^5\ N/mm^2 \cdot 1336 \cdot 10^4\ mm^4}{(1 \cdot 8400\ mm)^2} \cdot \left[\sqrt{\left(\frac{1}{1}\right)^2 \cdot \frac{10800 \cdot 10^6\ mm^6}{1336 \cdot 10^4\ mm^4} + \frac{(1 \cdot 8400\ mm)^2 \cdot 80000\ N/mm^2 \cdot 20{,}98 \cdot 10^4\ mm^4}{\pi^2 \cdot 2{,}1 \cdot 10^5\ N/mm^2 \cdot 1336 \cdot 10^4\ mm^4} + (1{,}562 \cdot 95\ mm)^2} - 1{,}562 \cdot 95\ mm\right] = 54325868{,}94\ Nmm$$

(4.3.53)

$$\lambda_{LT} = \sqrt{\frac{\omega_{pl,y} \cdot f_y}{M_{cr}}}$$

(4.3.54)

$$\lambda_{LT} = \sqrt{\frac{429500\ mm^3 \cdot 275 N/mm^2}{54{,}32 \cdot 10^6 Nmm}} = 1{,}4745$$

(4.3.55)

Table 6.3: Recommended values for imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

Image 33: Table of recommended values of imperfection coefficient for lateral buckling (EN 1993-1-1:2005)

Table 6.4: Recommended values for lateral torsional buckling curves for cross-sections using equation (6.56)

Cross-section	Limits	Buckling curve
Rolled I-sections	h/b ≤ 2 h/b > 2	**a** **b**
Welded I-sections	h/b ≤ 2 h/b > 2	**c** **d**
Other cross-sections	-	**d**

Image 34: Table of recommended values for lateral torsional buckling curves for cross-sections (EN 1993-1-1:2005)

$$\frac{h}{b} = \frac{190mm}{200mm} = 0{,}95 < 2$$

(4.3.56)

The selected curve is a with a value of $\alpha = 0{,}21$

$$\Phi = 0{,}5 \cdot (1 + \alpha_{LT} \cdot (\overline{\lambda_{LT}} - 0{,}2) + \overline{\lambda_{LT}}^2)$$

(4.3.57)

$$\Phi = 0{,}5 \cdot (1 + 0{,}21 \cdot (1{,}4745 - 0{,}2) + 1{,}4745^2) = 1{,}72$$

(4.3.58)

$$\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_{LT}}^2}}$$

(4.3.59)

$$\chi_{LT} = \frac{1}{1{,}72 + \sqrt{1{,}72^2 - 1{,}4745^2}} = 0{,}3834$$

(4.3.)

7. Profile resistance (6.3.3 EN 1993-1-1:2005)

$$\frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{LT} \cdot \omega_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{z,Ed}}{\frac{\omega_{pl,z} \cdot f_y}{\gamma_{M1}}} \leq 1{,}0$$

(4.3.60)

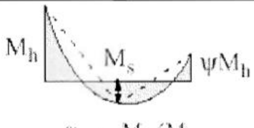
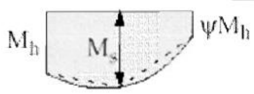
$$\frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} + k_{zy} \cdot \frac{M_{y,Ed}}{\frac{\chi_{LT} \cdot \omega_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{zz} \cdot \frac{M_{z,Ed}}{\frac{\omega_{pl,z} \cdot f_y}{\gamma_{M1}}} \leq 1{,}0$$

(4.3.61)

- $N_{Ed}, M_{y,Ed}\ and\ M_{z,Ed}$ - Design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively.
- $\chi_y\ and\ \chi_z$ - Reduction factors due to flexural buckling from 6.3.1.
- χ_{LT} - Reduction factor due to lateral torsional buckling from 6.3.2.
- $k_{yy},\ k_{yz}, k_{zy}, k_{zz}$ - Interaction factors.

According to table B.3 (EN 1993-1-1:2005):

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	C_{my} and C_{mz} and C_{mLT} concentrated load
	$-1 \le \psi \le 1$		$0{,}6 + 0{,}4\psi \ge 0{,}4$	
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$
	$-1 \le \alpha_s < 0$	$0 \le \psi \le 1$	$0{,}1 - 0{,}8\alpha_s \ge 0{,}4$	$-0{,}8\alpha_s \ge 0{,}4$
$\alpha_s = M_s/M_h$		$-1 \le \psi < 0$	$0{,}1(1-\psi) - 0{,}8\alpha_s \ge 0{,}4$	$0{,}2(-\psi) - 0{,}8\alpha_s \ge 0{,}4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
$\alpha_h = M_h/M_s$		$-1 \le \psi < 0$	$0{,}95 + 0{,}05\alpha_h(1+2\psi)$	AC2 $0{,}90 + 0{,}10\alpha_h(1+2\psi)$ AC2

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0{,}9$ or AC2 C_{mz} AC2 $= 0{,}9$ respectively.

C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:

moment factor	bending axis	points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

Image 35: Table of equivalent uniform moment factors Cm (EN 1993-1-1:2005)

The values of C_m are:

$$C_{my} = 0{,}95$$

$$C_{mz} = 0{,}95$$

$$C_{mLT} = 0{,}95$$

The iteration coefficients can be calculated following the table B.2 (EN 1993-1-1:2005):

$$k_{yy} = C_{my} \cdot \left(1 + \left(\overline{\lambda_y} - 0{,}2\right) \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}}\right) \le C_{my} \cdot \left(1 + 0{,}8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}}\right) \quad (4.3.62)$$

$$0{,}95 \cdot \left(1 + (1{,}174 - 0{,}2) \cdot \frac{20600\,N}{\frac{0{,}4925 \cdot 5380\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \leq 0{,}95 \cdot \left(1 + 0{,}8 \cdot \frac{26000\,N}{\frac{0{,}4925 \cdot 5380\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \tag{4.3.63}$$

$$0{,}976 \leq \mathbf{0,971}$$

$k_{zz} =$

$$C_{mz} \cdot \left(1 + \left(2 \cdot \overline{\lambda_z} - 0{,}6\right) \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}} \right) \leq C_{mz} \cdot \left(1 + 1{,}4 \cdot \frac{N_{Ed}}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}} \right) \tag{4.3.64}$$

$$0{,}95 \cdot \left(1 + (2 \cdot 1{,}952 - 0{,}6) \cdot \frac{20600\,N}{\frac{0{,}6579 \cdot 5380\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \leq 0{,}95 \cdot \left(1 + 1{,}4 \cdot \frac{20600\,N}{\frac{0{,}6579 \cdot 5380\text{mm}^2 \cdot 275\ \text{N/mm}^2}{1}} \right) \tag{4.3.65}$$

$$1{,}016 \leq \mathbf{0,978}$$

$k_{zy} =$

$$\left(1-\frac{0{,}1\cdot\overline{\lambda_z}}{(C_{mLT}-0{,}25)}\cdot\frac{N_{Ed}}{\frac{\chi_z\cdot A\cdot f_y}{\gamma_{M1}}}\right)\geq\left(1-\frac{0{,}1}{(C_{mLT}-0{,}25)}\cdot\frac{N_{Ed}}{\frac{\chi_z\cdot A\cdot f_y}{\gamma_{M1}}}\right) \tag{4.3.66}$$

$$\left(1-\frac{0{,}1\cdot 1{,}952}{(0{,}95-0{,}25)}\cdot\frac{20600\ N}{\frac{0{,}6579\cdot 5380\text{mm}^2\cdot 275\ \text{N/mm}^2}{1}}\right)\geq\left(1-\frac{0{,}1}{(0{,}95-0{,}25)}\cdot\frac{20600\ N}{\frac{0{,}6579\cdot 5380\text{mm}^2\cdot 275\ \text{N/mm}^2}{1}}\right) \tag{4.3.67}$$

$$0{,}994\geq\mathbf{0{,}997}$$

$k_{yz} =$

$$k_{yz}=0{,}6\cdot k_{zz}=\mathbf{0{,}585} \tag{4.3.68}$$

Finally:

$k_{yy} = 0{,}971$

$k_{zz} = 0{,}978$

$k_{zy} = 0{,}997$

$k_{yz} = 0{,}585$

$$\frac{20600\ N}{\frac{0{,}4925\cdot 5380\text{mm}^2\cdot 275\ \text{N/mm}^2}{1}}+0{,}971\cdot\frac{46{,}44\cdot 10^6\ N\cdot mm}{\frac{0{,}3834\cdot 429500\text{mm}^3\cdot 275\ \text{N/mm}^2}{1}}+0{,}585\cdot\frac{18{,}68\cdot 10^6\ N\cdot mm}{\frac{203800\text{mm}^3\cdot 275\ \text{N/mm}^2}{1}}=\mathbf{0{,}8758} \tag{4.3.69}$$

$$\mathbf{1{,}219} \not< \mathbf{1{,}0}$$

$$\frac{20600\,N}{\frac{0{,}4925 \cdot 5380\mathrm{mm}^2 \cdot 275\ \mathrm{N/mm}^2}{1}} + 0{,}997 \cdot \frac{46{,}44 \cdot 10^6\ N \cdot mm}{\frac{0{,}3834 \cdot 429500\mathrm{mm}^3 \cdot 275\ \mathrm{N/mm}^2}{1}} + 0{,}978 \cdot \frac{18{,}68 \cdot 10^6\ N \cdot mm}{\frac{203800\mathrm{mm}^3 \cdot 275\ \mathrm{N/mm}^2}{1}} = \mathbf{1{,}083}$$

(4.3.70)

$$\mathbf{1{,}376} \not< \mathbf{1{,}0}$$

The resistance conditions are not satisfied, the profile is not valid due to the lateral buckling, in order to reduce it, the beam should be braced reducing the L_u to the half.

$$M_{cr} = 1{,}285 \cdot \frac{\pi^2 \cdot 2{,}1 \cdot 10^5\ N/mm^2 \cdot 1336 \cdot 10^4\ mm^4}{(1 \cdot 4200\ mm)^2} \cdot \left[\sqrt{\left(\frac{1}{0{,}5}\right)^2 \cdot \frac{10800 \cdot 10^6\ mm^6}{1336 \cdot 10^4\ mm^4} + \frac{(1 \cdot 4200\ mm)^2 \cdot 80000\ N/mm^2 \cdot 20{,}98 \cdot 10^4\ mm^4}{\pi^2 \cdot 2{,}1 \cdot 10^5\ N/mm^2 \cdot 1336 \cdot 10^4\ mm^4} + (1{,}562 \cdot 95\ mm)^2} - 1{,}562 \cdot 95\ mm\right] = 226764561{,}1\ Nmm$$

(4.3.71)

$$\lambda_{LT} = \sqrt{\frac{429500\ mm^3 \cdot 275 N/mm^2}{226{,}76 \cdot 10^6 Nmm}} = 0{,}7217$$

(4.3.72)

$$\Phi = 0{,}5 \cdot (1 + 0{,}21 \cdot (0{,}7217 - 0{,}2) + 0{,}7217^2) = 0{,}8152$$

(4.3.73)

$$\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda_{LT}}^2}}$$

(4.3.74)

$$\chi_{LT} = \frac{1}{0{,}8152 + \sqrt{0{,}8152^2 - 0{,}7217^2}} = 0{,}8373$$

(4.3.75)

$$\frac{20600\ N}{\frac{0{,}4925\cdot 5380\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}971\cdot\frac{46{,}44\cdot 10^6\ N\cdot mm}{\frac{0{,}8373\cdot 429500\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}585\cdot\frac{18{,}68\cdot 10^6\ N\cdot mm}{\frac{203800\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}=\mathbf{0{,}7076}\tag{4.3.76}$$

$$\mathbf{0{,}6792<1{,}0}$$

$$\frac{20600\ N}{\frac{0{,}4925\cdot 5380\mathrm{mm}^2\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}997\cdot\frac{46{,}44\cdot 10^6\ N\cdot mm}{\frac{0{,}8373\cdot 429500\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}+0{,}978\cdot\frac{18{,}68\cdot 10^6\ N\cdot mm}{\frac{203800\mathrm{mm}^3\cdot 275\ \mathrm{N/mm}^2}{1}}=\mathbf{0{,}8739}\tag{4.3.77}$$

$$\mathbf{0{,}8224<1{,}0}$$

The resistance conditions are satisfied, the profile is OK.

The new disposition of the bridge will have a beam HEA 100 in the middle.

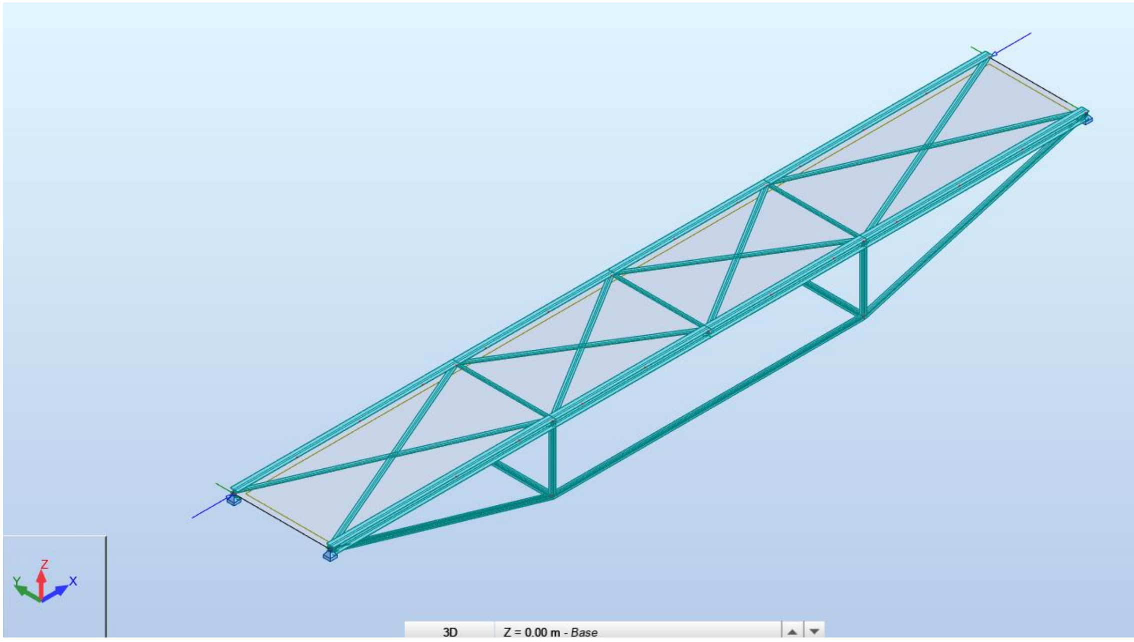


Image 36: New disposition of the bridge

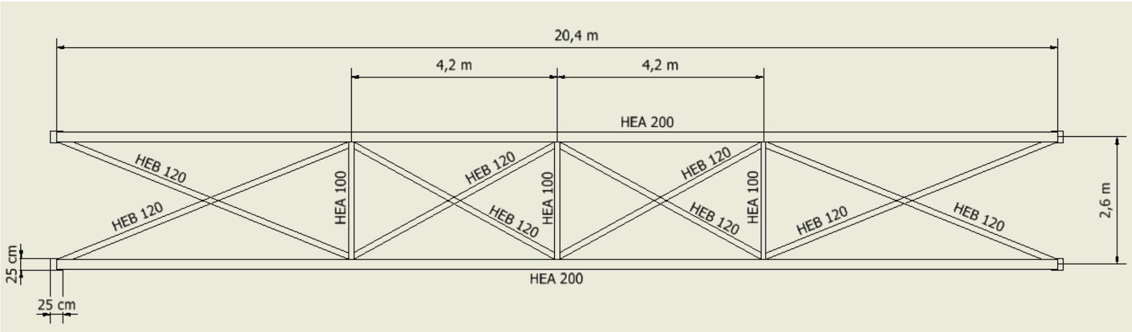


Image 37: Plan of the new disposition of the bridge

5 *FOUNDATION*

For structures like this one, strip footing is the most convenient type of foundation. It is going to be calculated as a separated individual support with the combination of the highest stresses.

The most unfavourable conditions for the foundation is calculated according to the results by AUTODESK ROBOT. The highest values of every type of reaction are considered

The following assumptions are going to be taken: the resistance of the soil is $0{,}2\ MPa$, the internal friction angle is 30º with no cohesion and the supports between the foundation and beams have an area of $25x25\ cm^2$.

Nudo/Caso	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
1/ 7 (C)	-353.44	-13.00	121.41	-0.00	0.00	-0.00
1/ 9 (C)	-298.71	-21.54	94.24	-0.00	0.00	-0.00
2/ 7 (C)	-290.86	-8.87	119.15	0.00	-0.00	0.00
1/ 8 (C)	-280.72	-12.97	94.32	-0.00	0.00	-0.00
2/ 8 (C)	-218.14	-8.91	92.05	0.00	-0.00	0.00
2/ 9 (C)	-194.41	-14.92	90.46	0.00	-0.00	-0.00
1/ 10 (C)	-127.91	-36.43	12.47	-0.00	-0.00	0.00
8/ 10 (C)	-60.59	-36.41	12.47	-0.00	0.00	-0.00
2/ 10 (C)	60.58	-36.40	12.47	0.00	-0.00	-0.00
5/ 10 (C)	127.92	-36.44	12.47	-0.00	0.00	-0.00
8/ 9 (C)	194.41	-14.92	90.46	-0.00	-0.00	0.00
8/ 8 (C)	218.14	-8.91	92.05	-0.00	0.00	0.00
5/ 8 (C)	280.72	-12.97	94.32	-0.00	0.00	-0.00
8/ 7 (C)	290.86	-8.87	119.15	-0.00	-0.00	0.00
5/ 9 (C)	298.71	-21.54	94.24	-0.00	0.00	-0.00
5/ 7 (C)	353.44	-13.00	121.41	-0.00	0.00	-0.00

Image 38: Table of reactions on supports

The reaction values are:

- $F_x = -353{,}44\ kN$
- $F_y = -36{,}44\ kN$
- $F_z = 121{,}44\ kN$
- $M_x = 0\ kNm$
- $M_y = 0\ kNm$
- $M_z = 0\ kNm$

Finally, the calculations are going to be done with these values:

- $N_p = 121{,}44\ kN$
- $V_{p,x} = 353{,}44\ \ kN$
- $V_{p,y} = 36{,}44\ kN$
- $M_{p,y} = 0\ kN \cdot m$
- $M_{p,x} = 0\ kN \cdot m$

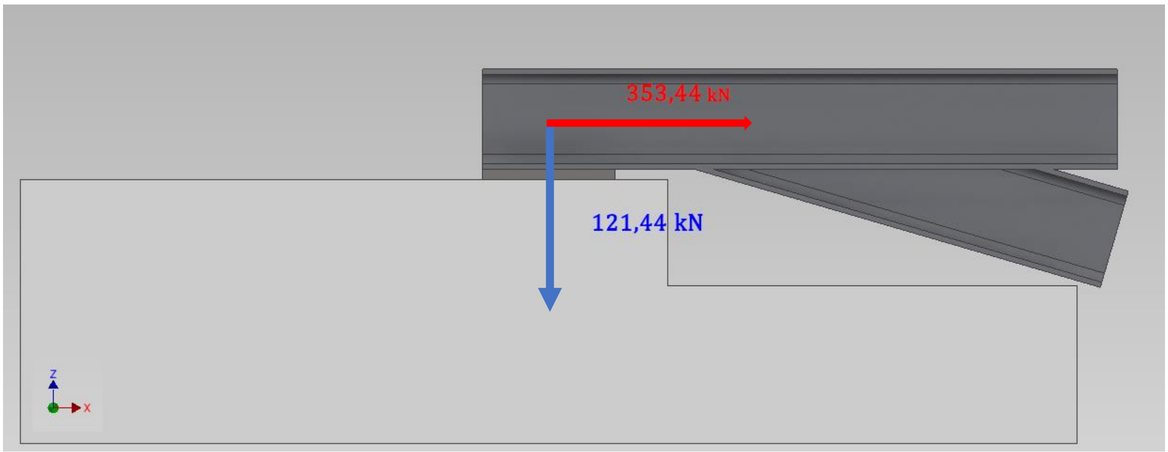


Image 39: Reactions and foundation drawing

1. U.L.S Subsiding

The estimated weight of the foundation is usually calculated as $W_f = 0{,}1 \cdot N_p$, but in this case, this value is going to be doubled:

$$W_f = 0{,}2 \cdot N_p$$

$$W_f = 0{,}2 \cdot 123{,}52\ kN \cong 25\ kN \tag{5.1}$$

The pressure transmitted by the foundation to the soil must be lower than the admissible one.

$$\sigma_{adm} = \frac{N_p}{a \cdot b} + \frac{M}{\omega} \tag{5.2}$$

The distance a is going to be taken almost as half width of the bridge plus 30 cm:

$$a = \frac{2{,}6\ m}{2} + 0{,}3\ m = 1{,}6\ m \tag{5.3}$$

$$b = \frac{N_p}{\sigma_{adm} \cdot a} = \frac{121{,}44\ kN + 25\ kN}{0{,}2\ N/mm^2 \cdot 1600\ mm} = 457\ mm \tag{5.4}$$

The relation between b/a should not be lower than 0,4 them the first approach will be:

$$1600\ mm\ x\ 700\ mm$$

2. Height

For flexible foundations:

$$v > 2 \cdot h$$

(5.5)

$$h < \frac{v}{2} = \begin{cases} \dfrac{a - a_p}{4} \\ \dfrac{b - b_p}{4} \end{cases}$$

(5.6)

$$h < \frac{v}{2} = \begin{cases} \dfrac{1600\ mm - 250\ mm}{4} = 337{,}5\ mm \\ \dfrac{700\ mm - 250\ mm}{4} = 112{,}5\ mm \end{cases}$$

(5.7)

The minimum height of the foundation according to the article 58.8 of EHE-08 is 250 mm but actually it should not be lesser than 0,5 m.

3. Resistance of the soil

First of all, the weight of the foundation has to be calculated:

$$W_f = a \cdot b \cdot h \cdot \rho_c$$

(5.8)

We can take the density of the soil as $\rho_c = 25\ kN/m^3$

$$W_f = 1{,}6\ m \cdot\ 0{,}7\ m \cdot 0{,}5\ m \cdot 25\ kN/m^3 = 14\ kN$$

(5.9)

The axial force will be:

$$N = 123{,}52\ kN + 14\ kN = 137{,}52\ kN$$

(5.10)

Because there is shear force, the bending moment will be:

$$M = M_p + V_p \cdot h$$

(5.11)

$$M_y = 0 + (-353{,}44\ kN) \cdot 0{,}5\ m = -176{,}72\ kNm$$

(5.12)

$$M_x = 0 + (-36{,}44\ kN) \cdot 0{,}5\ m = -18{,}22\ kNm$$

(5.13)

The eccentricity will be:

$$e_x = \frac{M_y}{N} = \frac{176{,}72 \cdot 10^6 N \cdot mm}{137{,}52 \cdot 10^3 N} = 1285{,}05\ mm$$

(5.14)

$$e_y = \frac{M_x}{N} = \frac{18{,}22 \cdot 10^6 N \cdot mm}{133{,}32 \cdot 10^3 N} = 136{,}66\ mm$$

(5.15)

The eccentricity is quite high due to the bending moment produced by the shear stresses in x direction and the height, the length in x direction (b) should be much higher.

The new values of foundation will be $1600\ mm\ x\ 2000\ mm\ x\ 500\ mm$
New weight of foundation:

$$W_f = 1{,}6\ m \cdot\ 2\ m \cdot 0{,}5\ m \cdot 25\ kN/m^3 = 40\ kN$$

(5.16)

New axial force:

$$N = 123{,}52\ kN + 40\ kN = 163{,}52\ kN$$

(5.17)

The pressure provided by the foundation to the floor is:

$$\sigma_0 = \frac{N}{a \cdot b} + \frac{M}{\omega} \leq \sigma_{adm}$$

(5.18)

$$M = F_z \cdot d = 121{,}44\ kN \cdot 0\ m = 0\ kNm$$

(5.19)

$$\omega = \frac{b \cdot h^2}{6} = 100 \cdot 10^6\ mm^3$$

(5.20)

$$\sigma_0 = \frac{163{,}52 \cdot 10^3\ N}{1600\ mm \cdot 2000\ mm} + \frac{0\ kN \cdot}{100 \cdot 10^6\ mm^3} = 0{,}0511\ MPa$$

(5.21)

$$0{,}0511\ Mpa < 0{,}2\ Mpa$$

(5.22)

The condition is fulfilled

4. U.L.S Landsliding

Landsliding in this structure can only be produced by the shear effect in “Y” direction (perpendicular to the direction of the bridge), and it is calculated according to the following equation.

$$F_d = \frac{V \cdot tg\Phi_c + B^* \cdot L^* \cdot c_c + R}{H} \geq 1{,}5 \tag{5.23}$$

Where:

- V - Effective vertical resultant.
- H - Resulting from the horizontal forces acting on the foundation plane.
- B^*, L^* - Dimensions of the equivalent rectangular foundation.
- Φ_c, c_c - Angle of friction and cohesion, of the contact of the foundation element.
- R - Sum of the possible additional resistances in the same direction and opposite direction to H.

For conventional concrete foundations:

$$tg\Phi_c = 0{,}8 tg\Phi \qquad\qquad c_c = c \tag{5.24}$$

Where:

- Φ_c - Angle of friction to consider in the ground-ground contact.
- c_c - Cohesion to consider in the ground-foundation contact.
- Φ - Angle of internal friction of the ground where the foundation supports.
- c - Cohesion of the ground where the foundation supports.

$$tg\Phi_c = 0{,}8 tg\ 30° \tag{5.25}$$

$$tg\Phi_c = 0{,}4618 \tag{5.26}$$

$$F_d = \frac{163{,}52 \cdot 10^3 N \cdot 0{,}4618}{36{,}74 \cdot 10^3 N} = 2{,}055 \tag{5.27}$$

$$2{,}055 \geq 1{,}5 \tag{5.28}$$

The condition is satisfied.

5. U.L.S rollover strength

The following condition has to be satisfied:

$$e \leq \frac{a}{4} \tag{5.29}$$

$$\frac{1600\,mm}{4} = 400\,mm \tag{5.30}$$

$$e_y = \frac{M_x}{N} = \frac{17{,}42 \cdot 10^6 N \cdot mm}{163{,}52 \cdot 10^3 N} = 106{,}52\,mm \tag{5.31}$$

$$106{,}52\,mm < 400\,mm \tag{5.32}$$

The condition is satisfied.

This condition does not need to be verified in “x” direction.

6. U.L.S structural resistance

For the dimensioning of the steel the weight of the foundation is not considered.

$$e_d = \frac{M_x}{N_p} = \frac{17{,}42 \cdot 10^6 N \cdot mm}{163{,}52 \cdot 10^3 N} = 106{,}52\,mm \tag{5.33}$$

The considered cover will be 50 mm, the useful height of the foundation will be:

$$d = h - r = 500\,mm - 50\,mm = 450\,mm \tag{5.34}$$

Steel parallel to side “a”

The traction force of the

$$T_{ad} = \frac{R_{1d}}{0{,}85 \cdot d} \cdot \left(x_1 - 0{,}25 \cdot a_p\right) = A_{s,a} \cdot f_{yd} \tag{5.35}$$

Where:

$$R_{1d} \cong \frac{N_d}{2 \cdot a} \cdot (a + 3 \cdot e_d) \tag{5.36}$$

$$x_1 = \frac{a}{4} \cdot \left(\frac{a + 4 \cdot e_d}{a + 3 \cdot e_d} \right) \tag{5.37}$$

The area of the steel will be:

$$A_{s,a} = \frac{R_{1d}}{0{,}85 \cdot d \cdot f_{yd}} \cdot (x_1 - 0{,}25 \cdot a_p) \tag{5.38}$$

Them:

$$R_{1d} \cong \frac{163{,}52 \cdot 10^3 \ N}{2 \cdot 1600 \ mm} \cdot (1600 \ mm + 3 \cdot 101{,}56 \ mm) \tag{5.39}$$

$$R_{1d} = 97{,}32 \ kN \tag{5.40}$$

$$x_1 = \frac{1600 \ mm}{4} \cdot \left(\frac{1600 \ mm + 4 \cdot 106{,}52 \ mm}{1600 \ mm + 3 \cdot 106{,}52 \ mm} \right) \tag{5.41}$$

$$x_1 = 422{,}19 \ mm \tag{5.42}$$

$$A_{s,a} = \frac{97{,}32 \ kN}{0{,}85 \cdot 450 \ mm \cdot 400 \ MPa/1{,}15} \cdot (422{,}19 \ mm - 0{,}25 \cdot 250 \ mm) \tag{5.43}$$

$$A_{s,a} = 263{,}11 \ mm^2 \tag{5.44}$$

The minimum amount of stell according to the table 42.3.5 of EHE-08 (page 201) is 0,0009 of the section for steel B500S.

$$\rho = \frac{A_{s,a}}{b \cdot h} \geq 0{,}0009 \tag{5.45}$$

$$A_{s,a} = 0{,}0009 \cdot 2600\, mm \cdot 500\, mm = 1170\, mm^2 \tag{5.46}$$

Finally:

$$A_{s,a} = 1170\, mm^2 \tag{5.47}$$

The diameter of the steel rots will be 12mm, them the number of rots:

$$n = 1 + \mathbb{N}\left(\frac{A_s}{\pi \cdot \frac{D^2}{4}}\right) \tag{5.48}$$

$$n = 1 + \left(\frac{1170mm^2}{\pi \cdot \frac{(12mm)^2}{4}}\right) = 11 \tag{5.49}$$

Distance between them:

$$s = \frac{b - 2 \cdot cover}{n - 1} \tag{5.50}$$

$$s = \frac{1600\, mm - 2 \cdot 50\, mm}{11 - 1} = 150\, mm \tag{5.51}$$

$\mathbf{11\emptyset 12}$ ***separate*** $\mathbf{15}$ ***cm***

Steel parallel to side "b"

The traction force of the

$$T_{bd} = \frac{N_d}{6{,}80 \cdot d} \cdot \left(b - b_p\right) = A_{s,a} \cdot f_{yd}$$

(5.52)

$$A_{s,a} = \frac{R_{1d}}{6{,}80 \cdot d \cdot f_{yd}} \cdot (b - b_p)$$

(5.53)

$$A_{s,b} = \frac{97{,}32 \cdot 10^3 N}{6{,}80 \cdot 450\, mm \cdot 400 MPa/1{,}15} \cdot (2000\, mm - 250\, mm)$$

(5.54)

$$A_{s,b} = 160\, mm^2$$

(5.55)

The minimum amount of stell according to the table 42.3.5 of EHE-08 (page 201) is 0,0009 of the section for steel B500S.

$$\rho = \frac{A_{s,b}}{b \cdot h} \geq 0{,}001$$

(5.56)

$$A_{s,a} = 0{,}0009 \cdot 2400\, mm \cdot 500\, mm = 1080\, mm^2$$

(5.57)

Finally:

$$A_{s,a} = 1080\, mm^2$$

(5.58)

The diameter of the steel rots will be 12 mm, them the number of rots:

$$n = 1 + \mathbb{N}\left(\frac{A_s}{\pi \cdot \frac{D^2}{4}}\right) \tag{5.59}$$

$$n = 1 + \left(\frac{1080\ mm^2}{\pi \cdot \frac{(12\ mm)^2}{4}}\right) = 10 \tag{5.60}$$

Distance between them:

$$s = \frac{b - 2 \cdot cover}{n - 1} \tag{5.61}$$

$$s = \frac{2000\ mm - 2 \cdot 50\ mm}{10 - 1} = 211{,}11\ mm \tag{5.62}$$

$\mathbf{10\emptyset 12}\ \boldsymbol{separate}\ \mathbf{20}\ \boldsymbol{cm}$

Actually, in order to facilitate the construction, the same configuration is going to be used in both directions.

The final configuration for both directions will be:

$$\mathbf{\emptyset 12}\ \boldsymbol{separate}\ \mathbf{15}\ \boldsymbol{cm}$$

The total volume of concrete used will be $1600mm\ x\ 2000mm\ x\ 500mm$ multiplied by the number of unions, in this case, four.

$$1{,}6\ m^3 \cdot 4 = 6{,}4\ m^3 \tag{5.63}$$

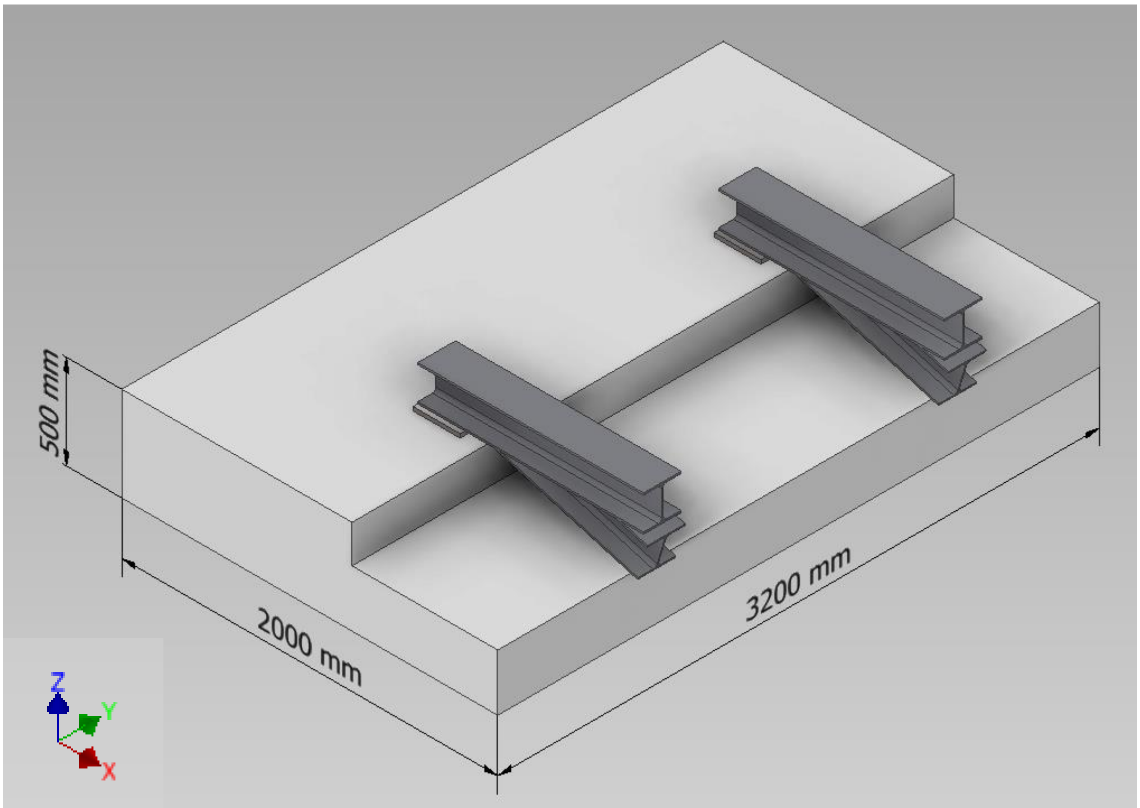


Image 40: Foundation drawing

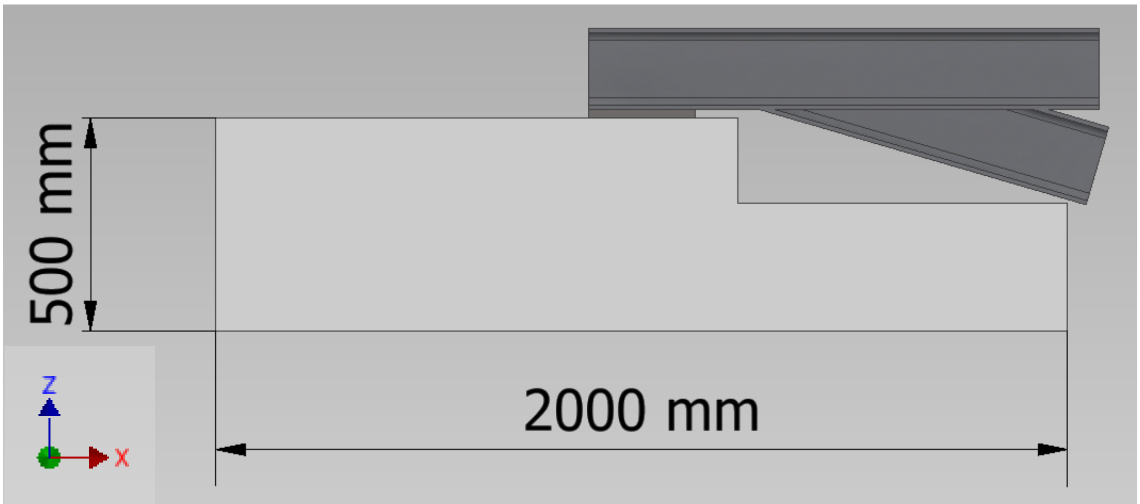


Image 41: Foundation drawing

6 DEFLECTIONS

According to standard, the maximum deflection in every beam should be lesser than $l/300$, this calculation is going to be done by Autodesk robot. First, combination of load should be done according to standards.

$$E_d = E\{ G_{k,j} \; ; \; P; \; Q_{kl} \; ; \; \Psi_{0i}; \; Q_{ki}\} \tag{6.1}$$

Where:

E_d - Project Load,

Ψ_{0i} – Reduction coefficient,

G_{KJ} – Permanent load,

Q_{KL} –Variable load,

P– Relevant representative value of a prestressing action.

$$E_d = 1{,}0 \cdot SW + 1{,}0 \cdot U + 1{,}0 \cdot SN + 1{,}0 \cdot W2 \tag{6.2}$$

Where:

- SW – Shelf Weight,
- U- Use,
- SN- Snow,
- W- Wind.

$$E_d = 1{,}0 \cdot SW + 1{,}0 \cdot U + 1{,}0 \cdot SN + 1{,}0 \cdot W2$$

$$E_d = 0{,}66 + 6{,}5 + 0{,}16 + 0{,}25 = 7{,}57 \; kN/m$$

Barra		Perfil	Material	Ratio(uy)	Caso (uy)	Ratio(uz)	Caso (uz)
20 Pandeo Lat +	OK	HEA 100	S 275	0.03	1*3 + 1*2 + 1*4	0.01	1*3 + 1*2 + 1*4
13 Pandeo Lat +	OK	HEA 100	S 275	0.03	1*3 + 1*2 + 1*4	0.01	1*3 + 1*2 + 1*4
44 Pandeo Lat +	OK	HEB 120	S 275	0.05	1*3 + 1*2 + 1*4	0.05	16 SLS
45 Pandeo Lat +	OK	HEB 120	S 275	0.05	1*3 + 1*2 + 1*4	0.05	16 SLS
46 Pandeo Lat +	OK	HEB 120	S 275	0.05	16 SLS	0.05	16 SLS
43 Pandeo Lat +	OK	HEB 120	S 275	0.05	16 SLS	0.05	16 SLS
22 Pandeo Lat +	OK	HEB 120	S 275	0.20	1*3 + 1*2 + 1*4	0.09	16 SLS
26	OK	HEB 120	S 275	0.20	1*3 + 1*2 + 1*4	0.09	16 SLS
16 Pandeo Lat +	OK	HEA 100	S 275	0.06	16 SLS	0.10	1*3 + 1*2 + 1*4
19 Pandeo Lat +	OK	HEA 100	S 275	0.06	16 SLS	0.10	1*3 + 1*2 + 1*4
9 Pandeo Lat + fl	OK	HEA 100	S 275	0.04	1*3 + 1*2 + 1*4	0.09	16 SLS
10 Pandeo Lat +	OK	HEA 100	S 275	0.04	1*3 + 1*2 + 1*4	0.09	16 SLS
18 Pandeo Lat +	OK	HEA 100	S 275	0.06	16 SLS	0.10	1*3 + 1*2 + 1*4
15 Pandeo Lat +	OK	HEA 100	S 275	0.06	16 SLS	0.10	1*3 + 1*2 + 1*4
4	OK	HEB 120	S 275	0.19	16 SLS	0.12	16 SLS
8 Pandeo Lat + fl	OK	HEB 120	S 275	0.19	16 SLS	0.12	16 SLS
17 Pandeo Lat +	OK	HEA 100	S 275	0.01	1*3 + 1*2 + 1*4	0.14	16 SLS
14 Barra 14	OK	HEA 100	S 275	0.05	1*3 + 1*2 + 1*4	0.15	16 SLS
38 Pandeo Lat +	OK	HEA 200	S 275	0.03	16 SLS	0.19	16 SLS
37 Pandeo Lat +	OK	HEA 200	S 275	0.03	16 SLS	0.19	16 SLS
40 Pandeo Lat +	OK	HEA 200	S 275	0.03	16 SLS	0.19	16 SLS
41 Pandeo Lat +	OK	HEA 200	S 275	0.03	16 SLS	0.19	16 SLS
21 Pandeo Lat +	OK	HEA 100	S 275	0.23	1*3 + 1*2 + 1*4	0.28	16 SLS
11 Pandeo Lat +	OK	HEA 100	S 275	0.23	1*3 + 1*2 + 1*4	0.28	16 SLS
23 Pandeo Lat +	OK	HEA 100	S 275	0.07	16 SLS	0.29	16 SLS
12 Pandeo Lat +	OK	HEA 100	S 275	0.07	16 SLS	0.29	16 SLS
42 Pandeo Lat +	OK	HEA 100	S 275	0.00	1*3 + 1*2 + 1*4	0.30	16 SLS
7 Pandeo Lat + fl	OK	HEA 200	S 275	0.33	16 SLS	0.32	16 SLS
5 Barra 5	OK	HEA 200	S 275	0.33	16 SLS	0.32	16 SLS
1 Barra 1	OK	HEA 200	S 275	0.34	16 SLS	0.33	16 SLS
3 Barra 3	OK	HEA 200	S 275	0.34	16 SLS	0.33	16 SLS

Image 42: Table of Steel profiles ordered by deflection.

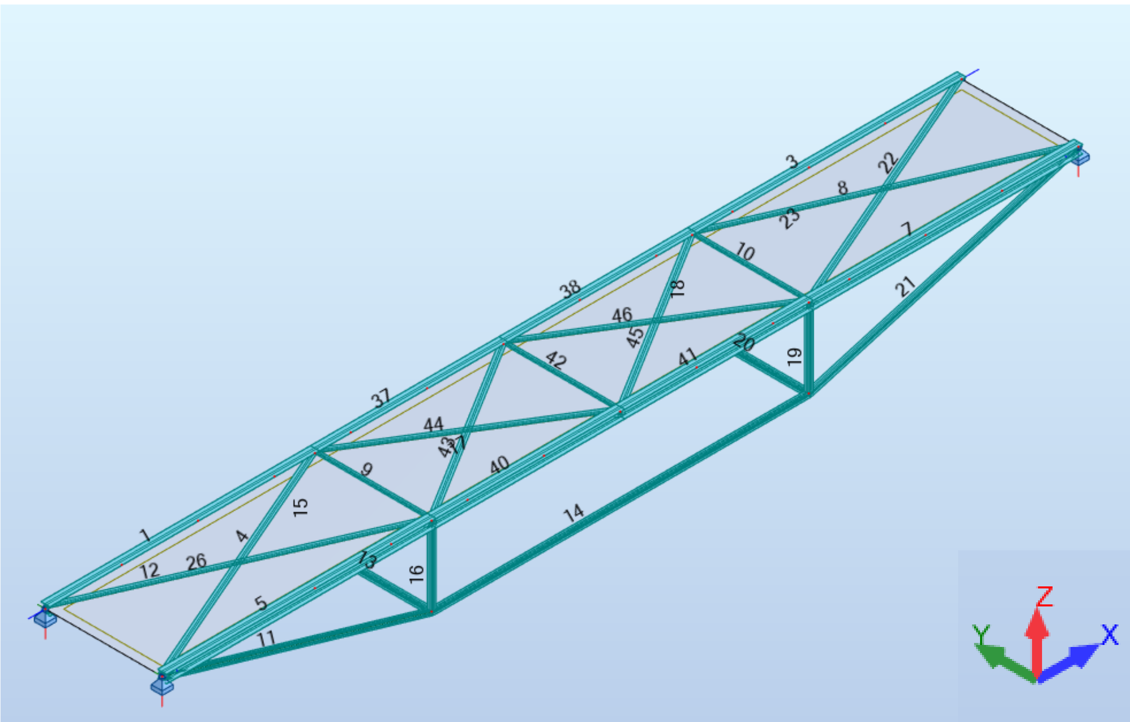


Image 43: Mentioned steel profiles.

Beam HEA 200 is the one with higher deflection, but it is still quite far away of the limit.

Verificado		
uz	-0.7	cm
uzt max(rel)	2.0	cm
uz max	2.0	cm
Solicit.(uz)	0.33	
Verificado		
u inst,z	-0.5	cm
u inst,max,z	2.0	cm
Solicit.(uinst,	0.26	

Image 44: Maximum deflection in HEA 200 and instant one.

7 MATERIAL COSTS

Table 6.1: Materials used

Profile	Quality	Weight (kg/m)	Total length (m)	Weight * length (kg)
HEA 200	S 275	42,3	40,8	1725,84
HEA 100	S 275	16,7	64,4	1075,48
HEB 120	S 275	26,7	87,94	2348,0
2,6 x 0,25 x 0,06 Wooden slab	C30	70,33	20,4	1434,72
6 mm steel wire	S 275	0,15	204	30,6

The costs are going to be calculated according to the tool "generadordeprecios" that provides the costs per kg of material taking account the indirect ones like transportation and installation.

Steel construction (S275): $5149{,}32\ kg\ x\ 1.72€/kg = 8856{,}83€$

Surface of Steel:
HEA200 $40{,}8m\ x\ 1{,}136\ m^2/m = 46{,}35\ m^2$
HEA100 $64{,}4m\ x\ 0{,}561\ m^2/m = 36{,}12\ m^2$
HEB120 $87{,}97m\ x\ 0{,}686\ m^2/m = 60{,}34\ m^2$

Corrosion protection: $142{,}81\ m^2\ x\ €20/m^2\ = 2856{,}35$ €

Wooden slabs: $1434{,}72\ kg\ x\ 1{,}91€/kg\ =\ 2740{,}31€$

Concrete based: $6{,}4\ m^3\ x\ 80€/m^3 = 512$ €

Steel wire: $204\ m\ x\ 0{,}80€/m = 163{,}2$ €

Total cost of the bridge material with corrosion protection is 15.123,68 €

8 CONCLUSIONS

This diploma thesis contains the design, calculation and dimensioning of a steel pedestrian bridge of a span 20,4 m according to the European Standards. The calculation of inter forces, deflections, dimensioning and 3D modeling has been done with Autodesk Robot. With the static analysis every profile has been checked according to the ultimate limit states (ULS), taking into account the shelf-weight, imposed load, wind load and snow load. The foundation has been calculated. Finally, the production cost of the bridge structure has been calculated: 15.123,68 €

9 SOURCES

EN 1991-1-1:2004//UNE-EN 1991-1-1:2019 Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight,imposed loads for buildings.

EN 1991-1-3:2004//UNE-EN 1991-1-3:2018 Eurocode 1 - Actions on structures - Part 1-3: General actions - Snow loads.

EN 1991-1-4:2005//UNE-EN 1991-1-4:2018 Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions.

EN 1993-1-1:2005//UNE-EN 1993-1-1:2008 Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings.

Real Decreto 1247/2008 instrucción de hormigón estructural (EHE-08).

Guía de cimentaciones en obras de Carretera.

10 ANNEX

10.1 LIST OF IMAGES

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Image 2: Lateral view.

Image 3: Perspective view.

Image 4: Lateral view.

Image 6: Rendered view with all the elements.

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Image 9: Weight of basic structural elements calculated by software.

Image 10: Map of snow zones AN.1 Spanish National Annex 4.1 (UNE-EN 1991-1-3:2018).

Image 11: Table of Sk values according to height Spanish National Annex 4.1 (UNE-EN 1991-1-3:2018).

Image 12: Directions of wind actions on bridges (EN 1991-1-4:2005// EN 1991-1-4:2005).

Image 13: Table of wind zones AN.1 Spanish National Annex 4.2 (UNE-EN 1991-1-4:2018).

Image 14: Coefficient of force for bridges, Cfx,0 (EN 1991-1-4:2005 (E)//UNE-EN 1991-1-4:2018).

Image 15: Table of terrain categories and terrain parameters.

Image 16: Exposure factor Ce(z) for Co=1,0, KI=1,0.

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Image 21: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005).

Image 22: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005).

Image 23: Table of recommended values of imperfection coefficient for lateral buckling (EN 1993-1-1:2005).

Image 24: Table of recommended values for lateral torsional buckling curves for cross-sections (EN 1993-1-1:2005).

Image 25: Table of equivalent uniform moment factors Cm (EN 1993-1-1:2005).

Image 26: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005).

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Image 28: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005).

Image 29: Table of recommended values for lateral torsional buckling curves for cross-sections (EN 1993-1-1:2005).

Image 30: Table of equivalent uniform moment factors Cm (EN 1993-1-1:2005).

Image 31: Table of imperfection coefficients for buckling curves (EN 1993-1-1:2005).

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Image 33: Table of recommended values of imperfection coefficient for lateral buckling (EN 1993-1-1:2005).

Image 34: Table of recommended values for lateral torsional buckling curves for cross-sections (EN 1993-1-1:2005).

Image 35: Table of equivalent uniform moment factors Cm (EN 1993-1-1:2005).

Image 36: New disposition of the bridge.

Image 37: Plan of the new disposition of the bridge.

Image 38: Table of reactions on supports.

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Table 3.1.2: Constant Weight of floor and fences

Table 3.4.1: Internal forces

Table 6.1: Materials used

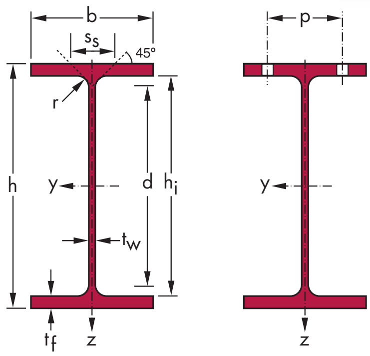
10.3 CHARACTERISTICS OF PROFILES

Perfiles H de alas anchas (Perfil europeo)

Dim.: HE A, HE B y HE M 100 - 1000 según la Euronorma 53-62; HE AA 100 - 1000; HL 920 - 1100
Tolerancias: EN 10034: 1993 HE 100 - 900; HE 1000 AA-M; HL AA-R
A6 - 05 HE con $G_{HE}>G_{HE\,M}$; HL 920; HL 1000 con $G_{HL}>G_{HL\,M}$
Estado de la superficie según norma EN 10163-3: 2004, clase C, subclase 1

European wide flange beams

Dim.: HE A, HE B and HE M 100 - 1000 in accordance with Euronorm 53-62; HE AA 100 - 1000; HL 920 - 1100
Tolerances: EN 10034: 1993 HE 100 - 900; HE 1000 AA-M; HL AA-R
A6 - 05 HE with $G_{HE}>G_{HE\,M}$; HL 920; HL 1000 with $G_{HL}>G_{HL\,M}$
Surface condition according to EN 10163-3: 2004, class C, subclass 1



Travi H ad ali larghe

Dimensioni: HE A, HE B e HE M 100 - 1000 in conformità con le Euronorm 53-62; HE AA 100 - 1000; HL 920 - 1100
Tolleranze: EN 10034: 1993 HE 100 - 900; HE 1000 AA-M; HL AA-R
A6 - 05 HE con $G_{HE}>G_{HE\,M}$; HL 920; HL 1000 con $G_{HL}>G_{HL\,M}$
Condizioni di superficie in conformità con EN 10163-3: 2004, classe C, sottoclasse 1

Denominación Designation Designazione	Dimensiones Dimensions Dimensioni						Dimensiones de construcción Dimensions for detailing Dimensioni di costruzione					Superficie Surface Superficie	
G kg/m	h mm	b mm	t_w mm	t_f mm	r mm	A mm^2	h_i mm	d mm	Ø	p_{min} mm	p_{max} mm	A_L m^2/m	A_G m^2/t
						x 10^2							
HE 100 AA• 12,2	91	100	4,2	5,5	12	15,6	80	56	M 10	54	58	0,553	45,17
HE 100 A 16,7	96	100	5	8	12	21,2	80	56	M 10	54	58	0,561	33,68
HE 100 B 20,4	100	100	6	10	12	26,0	80	56	M 10	56	58	0,567	27,76
HE 100 M 41,8	120	106	12	20	12	53,2	80	56	M 10	62	64	0,619	14,82
HE 120 AA• 14,6	109	120	4,2	5,5	12	18,6	98	74	M 12	58	68	0,669	45,94
HE 120 A 19,9	114	120	5	8	12	25,3	98	74	M 12	58	68	0,677	34,06
HE 120 B 26,7	120	120	6,5	11	12	34,0	98	74	M 12	60	68	0,686	25,71
HE 120 M 52,1	140	126	12,5	21	12	66,4	98	74	M 12	66	74	0,738	14,16
HE 140 AA• 18,1	128	140	4,3	6	12	23,0	116	92	M 16	64	76	0,787	43,53
HE 140 A 24,7	133	140	5,5	8,5	12	31,4	116	92	M 16	64	76	0,794	32,21
HE 140 B 33,7	140	140	7	12	12	43,0	116	92	M 16	66	76	0,805	23,88
HE 140 M 63,2	160	146	13	22	12	80,6	116	92	M 16	72	82	0,857	13,56
HE 160 AA• 23,8	148	160	4,5	7	15	30,4	134	104	M 20	76	84	0,901	37,81
HE 160 A 30,4	152	160	6	9	15	38,8	134	104	M 20	78	84	0,906	29,78
HE 160 B 42,6	160	160	8	13	15	54,3	134	104	M 20	80	84	0,918	21,56
HE 160 M 76,2	180	166	14	23	15	97,1	134	104	M 20	86	90	0,970	12,74
HE 180 AA• 28,7	167	180	5	7,5	15	36,5	152	122	M 24	84	92	1,018	35,51
HE 180 A 35,5	171	180	6	9,5	15	45,3	152	122	M 24	86	92	1,024	28,83
HE 180 B 51,2	180	180	8,5	14	15	65,3	152	122	M 24	88	92	1,037	20,25
HE 180 M 88,9	200	186	14,5	24	15	113,3	152	122	M 24	94	98	1,089	12,25
HE 200 AA• 34,6	186	200	5,5	8	18	44,1	170	134	M 27	96	100	1,130	32,62
HE 200 A 42,3	190	200	6,5	10	18	53,8	170	134	M 27	98	100	1,136	26,89
HE 200 B 61,3	200	200	9	15	18	78,1	170	134	M 27	100	100	1,151	18,78
HE 200 M 103	220	206	15	25	18	131,3	170	134	M 27	106	106	1,203	11,67
HE 220 AA• 40,4	205	220	6	8,5	18	51,5	188	152	M 27	98	118	1,247	30,87
HE 220 A 50,5	210	220	7	11	18	64,3	188	152	M 27	98	118	1,255	24,85
HE 220 B 71,5	220	220	9,5	16	18	91,0	188	152	M 27	100	118	1,270	17,77
HE 220 M 117	240	226	15,5	26	18	149,4	188	152	M 27	108	124	1,322	11,27

- Pedido mínimo: para calidad S 235 JR consular condiciones técnicas de suministro en página 218; para cualquier otra calidad 40 t o según acuerdo.
- Minimum order: for the S 235 JR grade cf. delivery conditions page 218; for any other grade 40 t or upon agreement.
- Ordine minimo: per il tipo S 235 JR vedere le condizioni tecniche di fornitura pagina 218; per qualsiasi altro tipo 40 t oppure secondo accordi.

HE

Páginas de notaciones 213-217 / Notations pages 213-217 / Pagine di annotazioni 213-217

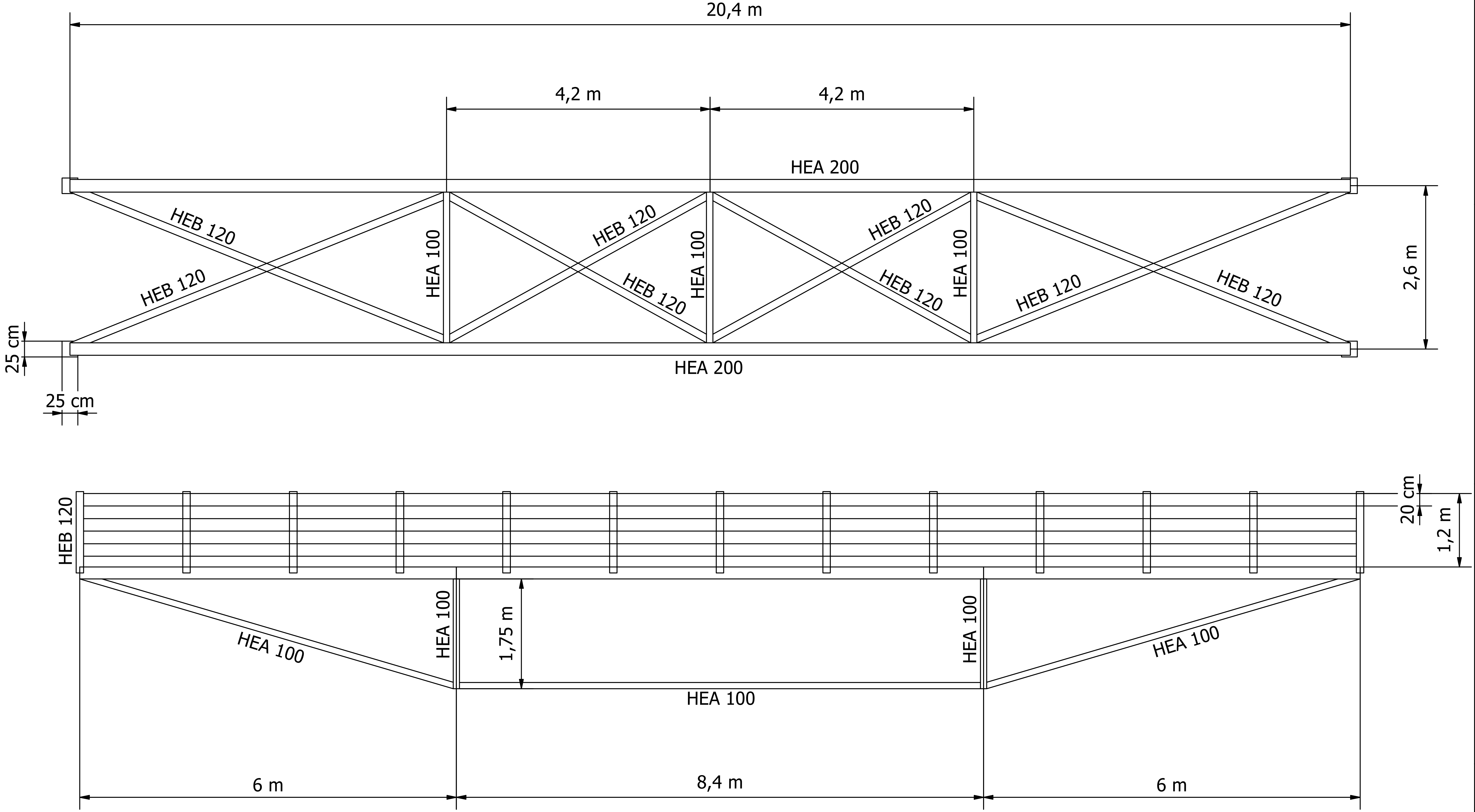
Denominación Designation Designazione	Propiedades del perfil / Section properties / Proprietà del profilato													Classification ENV 1993-1-1						EN 10025-2: 2004	EN 10025-4: 2004	EN 10225:2001
		eje fuerte y-y strong axis y-y asse forte y-y					eje débil z-z weak axis z-z asse debole z-z							pure bending y-y			pure compression					
G kg/m		I_y mm^4	$W_{el.y}$ mm^3	$W_{pl.y}$♦ mm^3	i_y mm	A_{vz} mm^2	I_z mm^4	$W_{el.z}$ mm^3	$W_{pl.z}$♦ mm^3	i_z mm	s_s mm	I_t mm^4	I_w mm^6	S 235	S 355	S 460	S 235	S 355	S 460			
		x 10^4	x 10^3	x 10^3	x 10	x 10^2	x 10^4	x 10^3	x 10^3	x 10		x 10^4	x 10^9									
HE 100 AA	12,2	236,5	51,98	58,36	3,89	6,15	92,06	18,41	28,44	2,43	29,26	2,51	1,68	1	3	3	1	3	3	✔	✔	✔
HE 100 A	16,7	349,2	72,76	83,01	4,06	7,56	133,8	26,76	41,14	2,51	35,06	5,24	2,58	1	1	1	1	1	1	✔	✔	✔
HE 100 B	20,4	449,5	89,91	104,2	4,16	9,04	167,3	33,45	51,42	2,53	40,06	9,25	3,38	1	1	1	1	1	1	✔	✔	✔
HE 100 M	41,8	1143	190,4	235,8	4,63	18,04	399,2	75,31	116,3	2,74	66,06	68,21	9,93	1	1	1	1	1	1	✔	✔	✔
HE 120 AA	14,6	413,4	75,85	84,12	4,72	6,90	158,8	26,47	40,62	2,93	29,26	2,78	4,24	2	3	4	2	3	4	✔	✔	✔
HE 120 A	19,9	606,2	106,3	119,5	4,89	8,46	230,9	38,48	58,85	3,02	35,06	5,99	6,47	1	1	2	1	1	2	✔	✔	✔
HE 120 B	26,7	864,4	144,1	165,2	5,04	10,96	317,5	52,92	80,97	3,06	42,56	13,84	9,41	1	1	1	1	1	1	✔	✔	✔
HE 120 M	52,1	2018	288,2	350,6	5,51	21,15	702,8	111,6	171,6	3,25	68,56	91,66	24,79	1	1	1	1	1	1	✔	✔	✔
HE 140 AA	18,1	719,5	112,4	123,8	5,59	7,92	274,8	39,26	59,93	3,45	30,36	3,54	10,21	3	3	4	3	3	4	✔	✔	✔
HE 140 A	24,7	1033	155,4	173,5	5,73	10,12	389,3	55,62	84,85	3,52	36,56	8,13	15,06	1	2	3	1	2	3	✔	✔	✔
HE 140 B	33,7	1509	215,6	245,4	5,93	13,08	549,7	78,52	119,8	3,58	45,06	20,06	22,48	1	1	1	1	1	1	✔	✔	✔
HE 140 M	63,2	3291	411,4	493,8	6,39	24,46	1144	156,8	240,5	3,77	71,06	120,0	54,33	1	1	1	1	1	1	✔	✔	✔
HE 160 AA	23,8	1283	173,4	190,4	6,50	10,38	478,7	59,84	91,36	3,97	36,07	6,33	23,75	3	3	4	3	3	4	✔	✔	✔
HE 160 A	30,4	1673	220,1	245,1	6,57	13,21	615,6	76,95	117,6	3,98	41,57	12,19	31,41	1	2	3	1	2	3	✔	✔	✔
HE 160 B	42,6	2492	311,5	354,0	6,78	17,59	889,2	111,2	170,0	4,05	51,57	31,24	47,94	1	1	1	1	1	1	✔	✔	✔
HE 160 M	76,2	5098	566,5	674,6	7,25	30,81	1759	211,9	325,5	4,26	77,57	162,4	108,1	1	1	1	1	1	1	✔	✔	✔
HE 180 AA	28,7	1967	235,6	258,2	7,34	12,16	730,0	81,11	123,6	4,47	37,57	8,33	46,36	3	3	4	3	3	4	✔	✔	✔
HE 180 A	35,5	2510	293,6	324,9	7,45	14,47	924,6	102,7	156,5	4,52	42,57	14,80	60,21	1	3	3	1	3	3	✔	✔	✔
HE 180 B	51,2	3831	425,7	481,4	7,66	20,24	1363	151,4	231,0	4,57	54,07	42,16	93,75	1	1	1	1	1	1	✔	✔	✔
HE 180 M	88,9	7483	748,3	883,4	8,13	34,65	2580	277,4	425,2	4,77	80,07	203,3	199,3	1	1	1	1	1	1	✔	✔	✔
HE 200 AA	34,6	2944	316,6	347,1	8,17	15,45	1068	106,8	163,2	4,92	42,59	12,69	84,49	3	4	4	3	4	4	✔	✔	✔
HE 200 A	42,3	3692	388,6	429,5	8,28	18,08	1336	133,6	203,8	4,98	47,59	20,98	108,0	1	3	3	1	3	3	✔	✔	✔
HE 200 B	61,3	5696	569,6	642,5	8,54	24,83	2003	200,3	305,8	5,07	60,09	59,28	171,1	1	1	1	1	1	1	✔	✔	✔
HE 200 M	103	10640	967,4	1135	9,00	41,03	3651	354,5	543,2	5,27	86,09	259,4	346,3	1	1	1	1	1	1	✔	✔	✔
HE 220 AA	40,4	4170	406,9	445,5	9,00	17,63	1510	137,3	209,3	5,42	44,09	15,93	145,6	3	4	4	3	4	4	✔	✔	✔
HE 220 A	50,5	5410	515,2	568,5	9,17	20,67	1955	177,7	270,6	5,51	50,09	28,46	193,3	1	3	3	1	3	3	✔	✔	✔
HE 220 B	71,5	8091	735,5	827,0	9,43	27,92	2843	258,5	393,9	5,59	62,59	76,57	295,4	1	1	1	1	1	1	✔	✔	✔
HE 220 M	117	14600	1217	1419	9,89	45,31	5012	443,5	678,6	5,79	88,59	315,3	572,7	1	1	1	1	1	1	✔	✔	✔

♦ W_{pl}: para el dimensionamiento plástico la sección debe pertenecer a la clase 1 o 2 según la capacidad de rotación que se precise. Ver página 217.

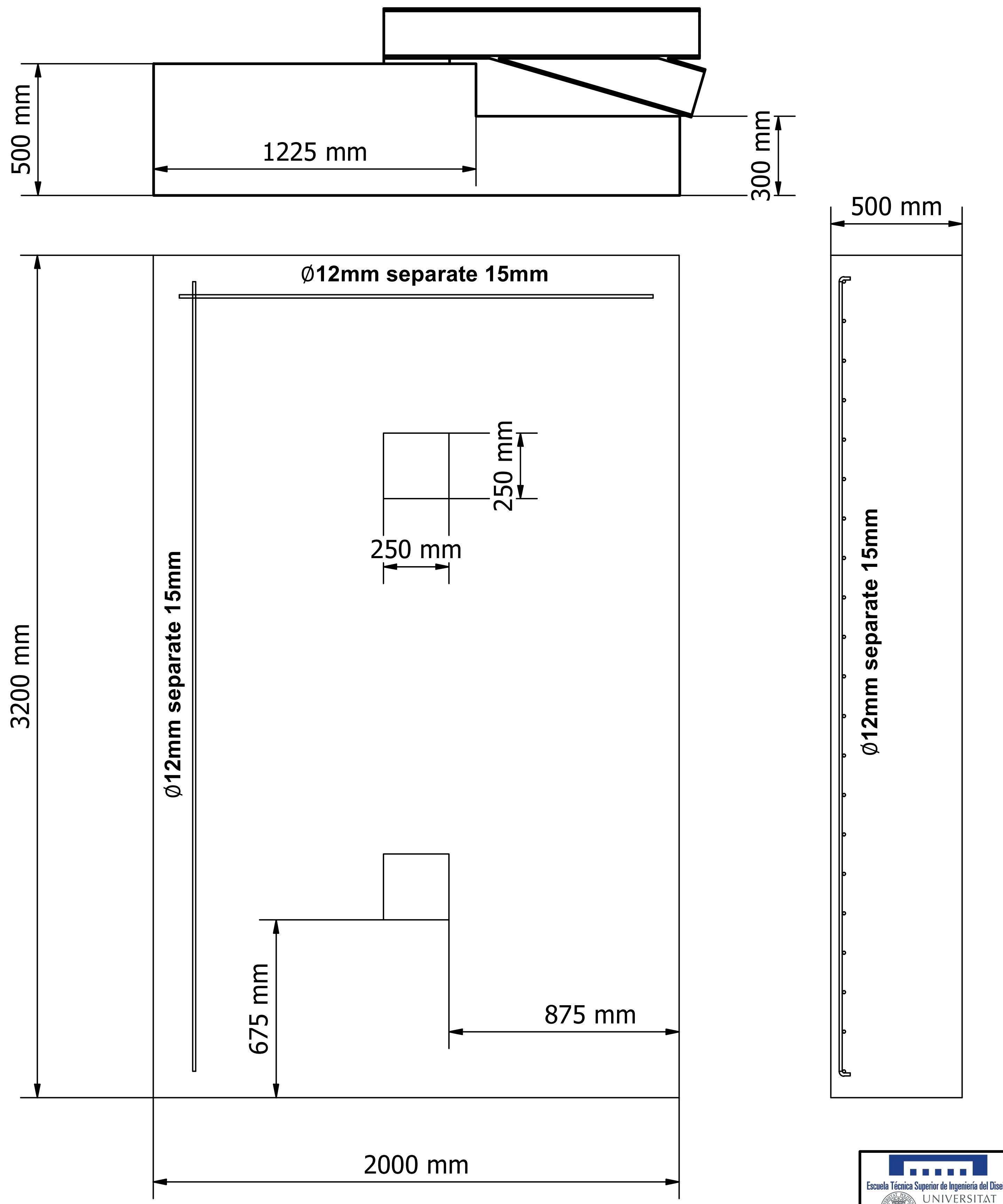
♦ W_{pl}: for plastic design, the shape must belong to class 1 or 2 according to the required rotation capacity. See page 217.

♦ W_{pl}: per dimensionamento alla rottura, la sezione deve appartenere alla classe 1 o 2 conformemente alla capacità di rotazione richiesta. Vedere pagina 217.

9.4 BUILDING PLANS



Escuela Técnica Superior de Ingeniería del Diseño — UNIVERSITAT POLITÈCNICA DE VALÈNCIA	Title: STATIC CALCULATION OF A 20,4 m STEEL BRIDGE FOR PEDESTRIAN USE.	
	Student: Pablo González Fernández	Date: August 2020
	Tutor: Stojan Kravanja // Teresa Real Herráiz	
Scale: 1 : 60	Sketch Name: Pedestrian Bridge	Nº of sheet: 1



Escuela Técnica Superior de Ingeniería del Diseño
UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Title: STATIC CALCULATION OF A 20,4 m STEEL BRIDGE FOR PEDESTRIAN USE.

Student: Pablo González Fernández

Tutor: Stojan Kravanja // Teresa Real Herráiz

Date: August 2020

Scale: 1 : 20

Sketch Name: Foundation

Nº of sheet: 2